CHAPTER 13

HORIZONTAL CONTROL

A system of control stations, local or universal, must be established to locate the positions of various points, objects, or details on the surface of the earth. The relative positions of detail points can be easily determined if these points are TIED IN to a local control station; or, if the control station is tied in to a geodetic control, the positions of other detail points can also be located relative to a worldwide control system.

The main control system is formed by a triangulation network supplemented by traverse. A traverse that has been established and is used to locate detail points and objects is often spoken of as a CONTROL TRAVERSE. Any line from which points and objects are located is a CONTROL LINE. A survey is controlled horizontally by measuring horizontal distances and horizontal angles. This type of survey is often referred to as HORIZONTAL CONTROL.

Horizontal control surveys are also conducted to establish supplementary control stations for use in construction surveys. Supplementary control stations usually consist of one or more short traverses run close to or across a construction area to afford easy tie-ins for various projects. These stations are established to the degree of accuracy needed for the purpose of the survey.

In this chapter, we will identify common procedures used in converting angular measurements taken from a compass or transit survey, recognize the methods used in establishing horizontal control, and identify various field procedures used in running a traverse survey.

DIRECTIONS AND DISTANCES

There are various ways of describing the horizontal locations of a point, as mentioned in chapter 12. In the final analysis, these ways are all reducible to the basic method of description; that is, by stating the length (distance) and direction of a straight line between the point whose location is being described and a reference point.

Direction, like horizontal location itself, is also relative; that is, the direction of a line can only be stated relative to a REFERENCE LINE of known (or sometimes of assumed) direction. In true geographical direction, the reference line is the meridian passing through the point where the observer is located; and the direction of a line passing through that point is described in terms of the horizontal angle between that line and the meridian. In magnetic geographical direction, the reference line is the magnetic meridian instead of the true meridian.

CONVERTING DIRECTIONS

The direction of a traverse line is commonly given by bearing. In field traversing, however, turning deflection angles with a transit is more convenient than orienting each traverse line to a meridian. The method of converting bearings to deflection angles is explained in the following paragraphs.

Converting Bearings to Deflection Angles

Converting bearings to deflection angles is based on the well-known geometrical proposition shown in figure 13-1.

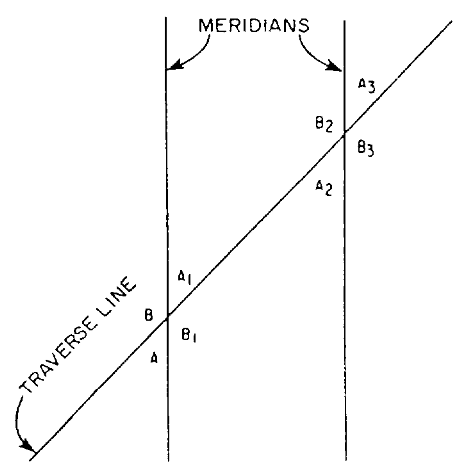


Figure 13-1.-Parallel lines (meridians) intersected by a traverse line, showing relationship of corresponding angles.

This figure shows two meridians or parallel lines that are intersected by another line called a traverse. It can be proved geometrically that the angles A and A_1, B and B_1, A_2 and A_3, and B_2 and B_3 are equal (vertically opposite angles). It can also be shown that angles $A = A_2$, and $B = B_2$ (corresponding angles). Therefore,

$$A = A_1 = A_2 = A_3 \text{ and}$$

$$B = B_1 = B_2 = B_3.$$

It can also be shown that the sum of the angles that form a straight line is 180°; the sum of all the angles around the point is 360°.

Figure 13-2 shows a traverse containing traverse lines AB, BC, and CD. The meridians through the traverse stations are indicated by the lines NS, N´S´, and N´´S´´. Although meridians are not, in fact, exactly parallel, they are assumed to be, for conversion purposes. Consequently, we have here three parallel lines intersected by traverses, and the angles created will therefore be equal, as shown in figure 13-1.

The bearing of AB is given as N20°E, which means that angle NAB measures 20°. To determine the deflection angle between AB and BC, you proceed as follows: If angle NAB measures 20°, then angle N'BB' must also measure 20° because the two corresponding angles are equal. The bearing of BC is given as S50°E, which means angle S´BC measures 50°E. The sum of angle N´BB´ plus S´BC plus the deflection angle between AB and BC (angle B´BC) is 180°. Therefore, the size of the deflection angle is

$$180° - (N'BB' + S'BC) \text{ or}$$

$$180° - (50° + 20°) = 110°.$$

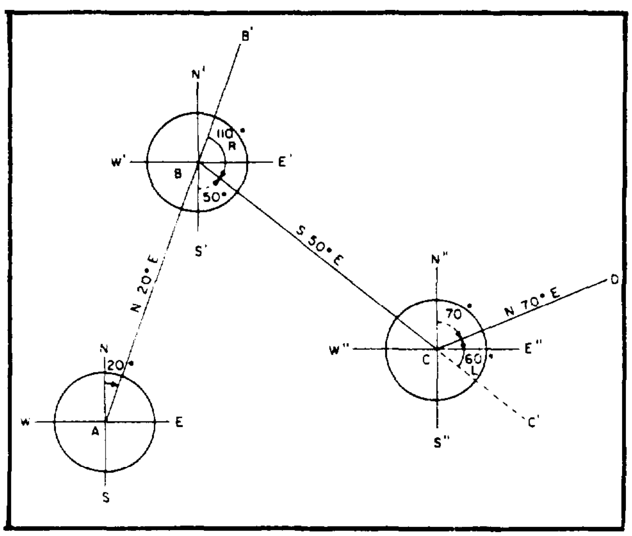


Figure 13-2.-Converting bearings to deflection angles from given traverse data.

The figure indicates that the angle should be turned to the right; therefore, the complete deflection angle description is 11°R.

The bearing of CD is given as N70°E; therefore, angle N´´CD measures70°. Angle S´´CC´ is equal to angle S´BC and therefore measures 50°. The deflection angle between BC and CD equals

$$180° - (S''CC + N''CD) \text{ or}$$

$$180° - (50° + 70°) = 60°.$$

The figure indicates that the angle should be turned to the left.

Converting Deflection Angles to Bearings

Converting deflection angles to bearings is simply the same process used for a different end result. Suppose that in figure 13-2, you know the deflection angles and want to determine the corresponding bearings. To do this, you must know the bearing of at least one of the traverse lines. Let's assume that you know the bearing of AB and want to determine the bearing of BC. You know the size of the deflection angle B´BC is 110°. The size of angle N´BB´ is the same as the size of NAB, which is 20°. The size of the angle of bearing of BC is

$$180° - (B'BC + NAB) \text{ or}$$

$$180° - (110° + 20°) = 50°.$$

The figure shows you that BC lies in the second or SE quadrant; therefore, the full description of the bearing is S50°E.

Converting Bearings to Interior and Exterior Angles

Converting a bearing to an interior or exterior angle is, once again, the same procedure applied for a different end result. Suppose that in figure 13-2, angle ABC is an interior angle and you want to determine the size. You know that angle ABS´ equals angle NAB, and therefore measures 20°.

You know from the bearing of BC that, angle S′BC measures 50°. The interior angle ABC is

$$ABS' + S'BC \text{ or}$$

$$20° + 50° = 70°.$$

The sum of the interior and exterior angles at any traverse station or point equals the sum of all the angles around that point, or 360°. Therefore, the exterior angle at station B equals 360° minus the interior angle or

$$360° - 70° = 290°.$$

The process of measuring angles around a point to obtain a check on their sum, which should equal 360°00', is sometimes referred to as CLOSING THE HORIZON.

Converting Azimuths to Bearings or Vice Versa

Suppose you want to convert an azimuth of 135° to the corresponding bearing. This azimuth is greater than 90° but less than 180°; therefore, the line lies in the southeast quadrant. As shown in figure 13-3, the bearing angles are always measured from the north and south ends of thereference meridian. (When solving any bearing problem, draw a sketch to get a clear picture.) For the azimuth, the horizontal direction is reckoned clockwise from the meridian plane. It is measured between either the north or the south end of the reference meridian and the line in question. When we talk about azimuth in this training manual, however, you must understand that the azimuth is referenced clockwise from the NORTH point of the meridian. The numerical value of this 135° azimuth angle is measured from the north. Therefore, in this figure, the value of the bearing is

$$180° - 135 = 45°.$$

The complete description of the bearing then is S45°E.

For example, if you want to convert a bearing of N30°W into an azimuth angle, you know that the angle location must be in the northwest quadrant. Then, draw an angle of 30° from the north end of the reference meridian because you measure azimuth angles clockwise from the north end of the reference meridian. To compute this azimuth angle, subtract 30° from 360°; the result is 330°. Therefore, the bearing of N30°W is equal to 330° azimuth angle.

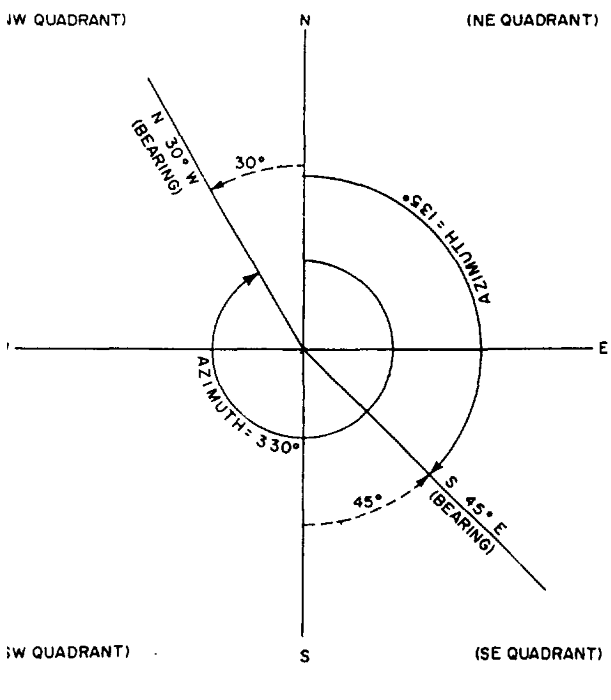


Figure 13-3.-Converting azimuths to corresponding bearings or vice versa.

ESTABLISHING DIRECTION BY SURVEYOR'S COMPASS

The basic method of establishing direction of a survey line or a point is with a surveyor's compass. (Notice that on most surveyor's compasses, the east and west indicators are in the opposite positions from those of the east and west indicators on a map or chart.) In figure 13-4, an

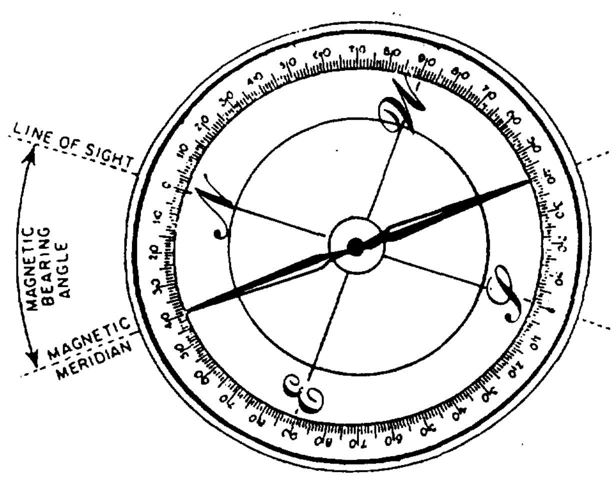


Figure 13-4.-A magnetic compass reading corrected for local attraction.

observer is determining the magnetic bearing of the dotted line labeled Line of Sight. First, the observer mounts the compass on a steady support, levels it, and waits for the needle to stop oscillating. Then, the observer carefully rotates the compass until the north-south line on the card lies exactly along the line whose bearing is being taken.

The bearing is now indicated by the needlepoint. The needlepoint indicates a numerical value of 40°. The card indicates the northeast quadrant. The magnetic bearing is, therefore, N40°E.

Correcting for Local Magnetic Attraction

Figure 13-4 shows the compass needle lying along the magnetic meridian. This means either that the compass is in an area free of "local magnetic attraction" or that the effect of local attraction has been eliminated by adjusting the compass card as described later. "Local magnetic attraction" means the deflection of the compass needle by a local magnetic force, such as that created by nearby electrical equipment or by a mass of metal, such as a bulldozer. When local attraction exists and is not compensated for, the bearing you get is a COMPASS bearing. A compass bearing does not become a magnetic bearing until it has been corrected for local attraction. Suppose, for example, you read a compass bearing of N37°E. Suppose the effect of the magnetic attraction of a nearby pole transformer is enough to deflect the compass needle 4° to the west of the magnetic meridian. In the absence of this local attraction, the compass would read N33°E, not N37°E. Therefore, the correct magnetic bearing is N33°E.

To correct a compass bearing for local attraction, you determine the amount and direction (east or west) of the local attraction. First, set up the compass where you propose to take the bearing. Then, select a distant object that you may presume to be outside the range of any local attraction. Take the bearing of this object. If you read a bearing of S60°W, shift the compass to the immediate vicinity of the object you sighted on; and take, from there, the bearing of the original setup point. In the absence of any local attraction at the original setup point, you would read the back bearing of the original bearing or N60°E. Suppose instead you read N48°E. The back bearing of this is S48°W. Therefore, the bearing as indicated by the compass under local attraction is S60°W; but as indicated by the compass not under local attraction, it is S48°W. The amount and direction of local attraction are, therefore, 12°W.

The question of whether you add the local attraction to, or subtract it from, the compass bearing to get the magnetic bearing depends on (1) the direction of the local attraction and (2) the quadrant the bearing is in.

As a rule, for a bearing in the <u>northeast quadrant, you add an easterly attraction to the compass bearing to get the magnetic bearing and subtract a westerly attraction from the compass bearing to get the magnetic bearing.</u>

Now, consider the compass shown in figure 13-5. This compass indicates a bearing of S40°W. Suppose the local attraction is 12°W. The needle, then, is 12°W of where it would be without local attraction. You can see that, <u>in the southwest quadrant, you would subtract westerly attraction and add easterly attraction.</u>

From a study of the paragraphs above, it becomes obvious that the procedure is the opposite for bearings in the northwest or southeast

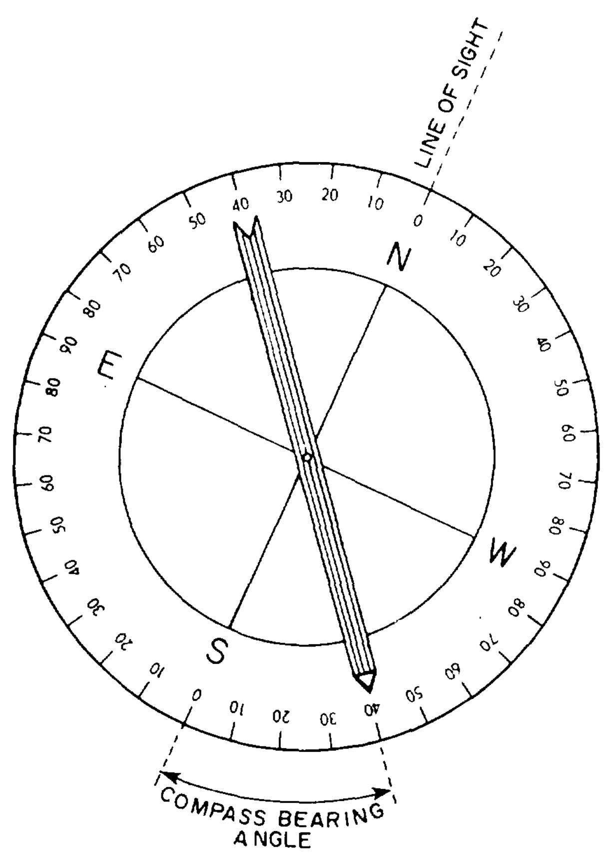


Figure 13-5.-Compass bearing affected by local magnetic attraction.

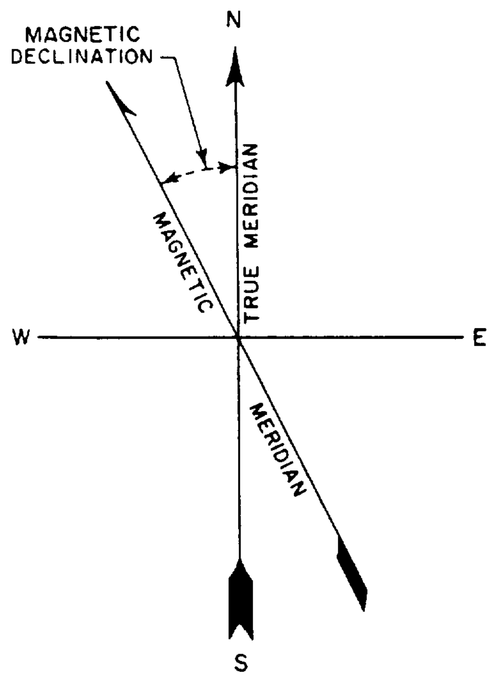


Figure 13-6.-Magnetic declination (west).

quadrants. In these quadrants, you add westerly attraction and subtract easterly attraction to the compass bearing to get the magnetic bearing.

Determining Magnetic Declination and Dip

The angle between the true meridian and the magnetic meridian is MAGNETIC DECLINATION. If the north end of the compass needle is pointing to the east of the true meridian, the declination is said to be east. If the north end of the compass needle is pointing to the west of the true meridian, the declination is said to be west. (See fig. 13-6.)

The magnetic needle aligns itself with the earth's magnetic field and points toward the earth's magnetic pole. In horizontal projections, these lines incline downward toward the north in the Northern Hemisphere and downward toward the south in the Southern Hemisphere. Since the bar takes the position parallel with the lines of force, it inclines with the horizontal. This phenomenon is the MAGNETIC DIP.

Converting Magnetic Bearings to True Bearings

When you have corrected a compass bearing for local attraction, you have a MAGNETIC BEARING. As explained previously, in most areas of the earth, a magnetic bearing differs from a true bearing by the amount of the local magnetic declination (called magnetic variation by navigators). The amount and direction of local declination are shown on maps or charts of the area in a format similar to the following: "Magnetic Declination 26°45°W (1968), Annual Increase 11′. " This means, if you are working in 1988 (20 years later), the local declination is

$$26°45' + (11' \times 20) \text{ or}$$

$$26°45' + 220' = 26°45' + 3°40' = 30°25'.$$

To convert a magnetic bearing to a TRUE BEARING, you apply the declination to the magnetic bearing in precisely the same way that you apply local attraction to a compass bearing. If the declination is east, it is added to northeast and southwest magnetic bearings, and it is subtracted from southeast and northwest magnetic bearings. If the declination is west, it is added to southeast and northwest magnetic bearings and subtracted from northeast and southwest magnetic bearings.

When you have a compass bearing and know both the local attraction and the local declination, you can go from compass bearing to true bearing in a single process by applying the ALGEBRAIC SUM of local attraction and local declination, Suppose that local attraction is 6°W and declination, 15°E. You could correct for local attraction and convert from magnetic to true in the same operation by applying a correction of 9°E to the compass bearing.

Uncorrecting and Unconverting

You correct a compass bearing to a magnetic bearing by applying the local attraction. You convert a magnetic bearing to a true bearing by applying the local declination.

At some time, you may be given a magnetic bearing and have to figure the corresponding compass bearing by using both local attraction and local declination.

The terms used to describe these calculations are, for the want of any better expressions, UNCORRECTING and UNCONVERTING. All YOU need to remember is that, when you are uncorrecting or unconverting, you apply local attraction and local declination in the REVERSE of the directions in which you apply them if you were correcting or converting.

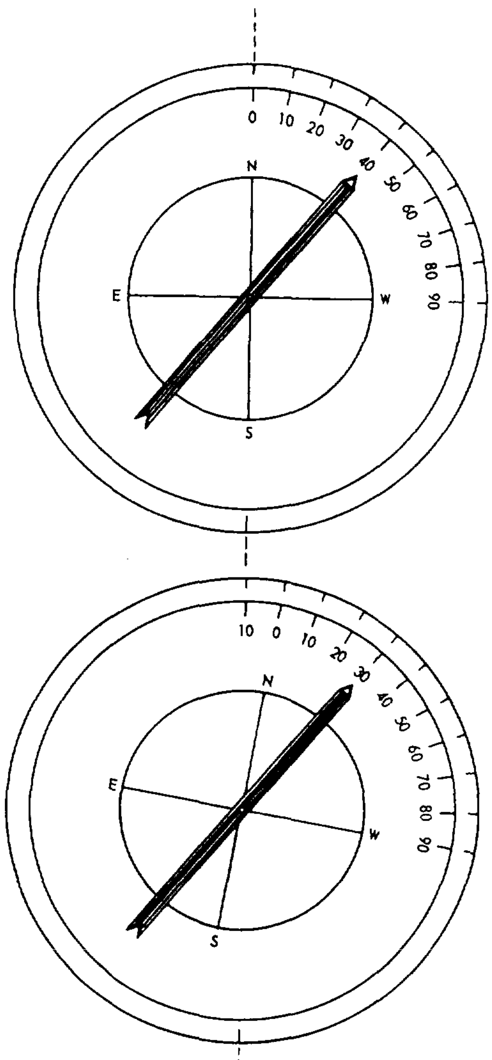


Figure 13-7.-Orienting a compass for a 10° easterly attraction.

For example, with a compass affected by a 10°W local attraction, you want to lay off a line bearing S28°W magnetic by compass. If you were correcting, you would subtract a westerly attraction in the southwest quadrant. However, for uncorrecting you ADD a westerly attraction in that quadrant. Therefore, to lay off a line bearing S28°W, you would lay off S38°W by the compass.

The same rule applies to azimuths. Suppose you have an azimuth-reading (measured from north) compass set up where local attraction is 10°W and declination is 25°E, and you want to lay off a line with true azimuth 256°. The algebraic sum of these is 15°E. For correcting or converting azimuths, you ADD easterly and SUBTRACT westerly corrections; therefore, if you were correcting or converting, you would add the 15° to 256°. Because you are uncorrecting or unconverting, however, you subtract; and, to lay off a line with true azimuth 256°, you read 241° by the compass.

Orienting a Compass

Some transit compasses and most surveyor's and forester's field compasses are equipped for offsetting local attraction, local declination, and/or the algebraic sum of the two. In figure 13-7, the upper view shows a compass bearing of N40°W on a compass presumed to be affected by a local attraction of 10°E. In this quadrant, you subtract easterly attraction; therefore, the magnetic bearing should read N30°W.

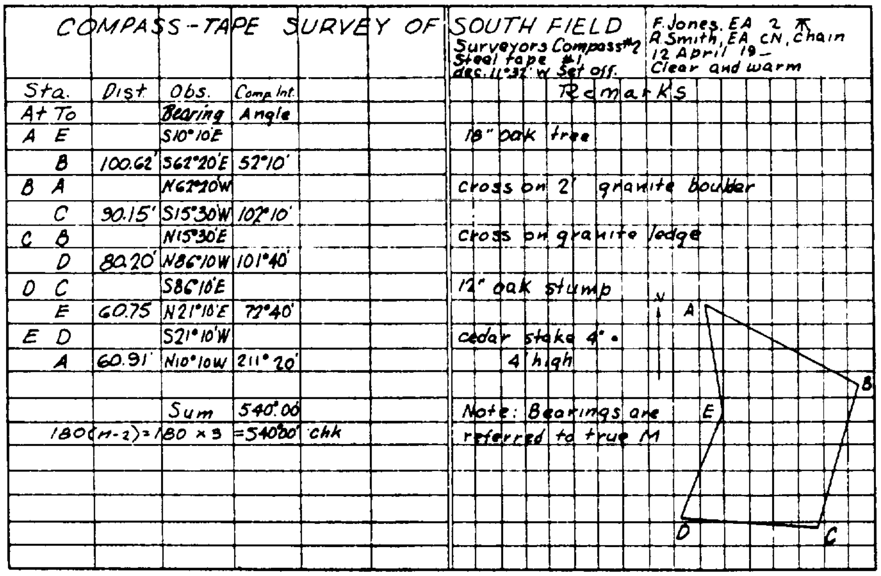
In the lower view, the same compass has been oriented for an error of 10°E by simply rotating the compass card 10°E clockwise. On most orienting compasses, the card can be released for rotating by backing off a small screw on the face of the card. Note that you now read the correct magnetic bearing of N30°W.

Conducting a Compass-Tape Survey

Figure 13-8 shows field notes from a compass-tape survey of a small field. The instrument used was a surveyor's compass. The compass was first set up at station A, shown in the sketch drawn on the remarks page. The first bearing taken was that of the line AE. This was actually the back bearing of EA, taken for the purpose of later checking against the forward bearing of EA.

Next, the bearing of AB was taken, and the distance from A to B was chained. The observed bearing (S62°20´E) was entered beside B in the column headed "Obs. Bearing." The chained distance was entered beside B in the column headed "Dist."

The compass was shifted to station B, and the back bearing of AB (that is, the bearing of BA) was taken as a check on the previously taken bearing of AB. The back bearing turned out to have, as it should have, the same numerical value (62°20´) as the forward bearing. A difference in the two would indicate either an inaccuracy in reading one bearing or the other or a difference in the strength of local attraction.



COMPASS-TAPE SURVEY OF SOUTH FIELD

Surveyors Compass #2
Steel tape #1
dec. 11°32′ W Set off.

F. Jones, EA 2 ⊼
R. Smith, EA CN, Chain
12 April 19—
Clear and warm

Sta. At	Sta. To	Dist	Obs. Bearing	Comp. Int. Angle	Remarks
A	E		S10°10′E		18″ oak tree
	B	100.62′	S62°20′E	52°10′	
B	A		N62°20′W		cross on 2′ granite boulder
	C	90.15′	S15°30′W	102°10′	
C	B		N15°30′E		cross on granite ledge
	D	80.20′	N86°10′W	101°40′	
D	C		S86°10′E		12″ oak stump
	E	60.75	N21°10′E	72°40′	
E	D		S21°10′W		cedar stake 4″ • 4′ high
	A	60.91′	N10°10′W	211°20′	
			Sum	540°00′	Note: Bearings are referred to true M
180(n-2) = 180 × 3				= 540°00′ chk	

Figure 13-8.-Sample field notes from a compass-tape survey.

Proceeding in this fashion, the party took bearings and back bearings, and chained distances all the way around to the starting point at station A. The last forward bearing taken, that of EA, has the same numerical value as the back bearing of EA (bearing of AE) taken at the start.

Checking Accuracy of Observed Bearings

As a check on the accuracy of the whole bearing-reading process, the size of the interior angle at each station was computed from the observed bearings by the method previously described for converting bearings to interior angles. The sizes of these angles were entered in the column headed "Comp. Int. Angle," and the sum was entered below.

The sum of the interior angles in a closed traverse should equal the product of 180° (n – 2), n being the number of traverse lines in the traverse. In this case, the traverse has five lines; therefore, the sum of the interior angles should be

$$180°(5 - 2) = 180° \times 3 = 540°.$$

The computed sum is, therefore, the same as the added sum of the angles converted from observed bearings.

Recognizing, Reducing, and Correcting Compass Errors

If a magnetic compass has a bent needle, there will be a constant instrumental error in all observed bearings and azimuths. To check for this condition, set up and level the compass, wait for the needle to cease oscillating, and read the graduation indicated at each end of the needle. If the compass is graduated for bearings, the numerical value at each end of the needle should be the same. If the compass is graduated for azimuths, the readings should be 180° apart.

Similarly, if the pivot supporting the needle on a magnetic compass is bent, there will be an instrumental error in the compass. However, this error, instead of being the same for all readings, will be variable.

You, can eliminate either of these instrumental errors by reading both ends of the needle and using the average between them. Suppose, for example, that with a compass graduated for bearings you read a bearing of N45°E and a back bearing of S44°W. You would use the average, or

$$1/2\ (45° + 44°) = N44°30'E.$$

The error in the compass should, of course, be corrected as soon as possible. Normally, this is a job for an expert. Remember the cause of a discrepancy in the reading at both ends when there is one. It is more probable that the needle, rather than the pivot, is bent. After a bent needle has been straightened, if a discrepancy still exists, then probably the pivot is bent too.

If a compass needle is sluggish—that is, if it moves unusually slowly in seeking magnetic north—it will probably come to rest a little off the magnetic meridian. The most common cause of sluggishness is weakening of the magnetism of the needle. A needle may be demagnetized by drawing it over a bar magnet. The needle should be drawn from the center of the bar magnet toward the end, with the south end of the needle drawn over the north end of the magnet and vice versa. On each return stroke, lift the needle well clear of the magnet.

Sometimes the cause of a sluggish needle is a blunt point on the pivot. This may be corrected by sharpening the pivot with a fine file.

If the compass is not level when a bearing or azimuth is being read, the reading will be incorrect. A similar error will exist if the compass is equipped with sighting vanes and one or more of them are bent. To check for bent compass vanes, you set up and level the compass, and then sight with the vanes on a plumb bob cord.

The most common personal error the observer can make in compass work is MISREADING. This is caused by the observer's eye not being vertically above the compass at the time of the reading. Other common mistakes are reading a needle at the wrong end and setting off local attraction or declination in the wrong direction when the compass is being oriented.

ESTABLISHING DIRECTIONS BY TRANSIT

Directions are similarly determined by the use of a transit. This can be done by measuring the size of the horizontal angle between the line whose direction is sought and a reference line. With a transit, however, you are expected to do this with considerably more accuracy and precision than with a surveyor's compass. Some of the basic procedures associated with the proper operation of the instrument will be discussed in the following paragraphs.

Setting Up the Transit

The point at which the line of sight, the horizontal axis, and the vertical axis of a transit meet is called the INSTRUMENT CENTER. The point on the ground over which the center of the instrument is placed is the INSTRUMENT POINT, TRANSIT POINT, or STATION. A wooden stake or hub is usually marked with a tack when used as a transit station or point. To prevent jarring or displacement of the transit, avoid those stations having loose planking, those having soft or marshy ground, and those having other conditions that would cause the legs of the tripod to move. The following steps are recommended when you are setting up a transit over a station point:

1. Center the instrument as closely as possible over the definite point by suspending a plumb line from a hook and chain beneath the instrument. The plumb string is tied with a slipknot, so that you can adjust the height of the plumb.
2. Move the tripod legs as necessary until the plumb bob is about 1/4 in. short of being over the tack, meanwhile maintaining a fairly level foot plate. Spread the tripod legs, and apply sufficient pressure to the legs to make sure of their firmness in the ground. Make sure to loosen the wing nuts to rid the static pressure in them before retightening.
3. Turn the plates so that each plate level is parallel to a pair of opposite leveling screws. (It is common practice to have a pair of opposite leveling screws in line with the approximate line of sight.) The leveling screws are then tightened to firmness, but not tight. Rotate opposing pairs of leveling screws either toward each other or away from each other until the plate bubbles are centered.

If the plumb bob is not directly over the center of the tack, you may loosen two adjacent leveling screws enough to free the shifting plate. Relevel the instrument if the bubbles become off-center. During breezy conditions, you may shield the plumb line with your body when setting up an instrument. Sometimes in windy locations, it may be necessary to construct a wind shield.

Setting and leveling the transit rapidly requires a skill on your part that you will learn and develop through consistent practice. You should take advantage of any opportunity that you can to train yourself and increase your skills in handling surveying instruments. Again, when setting up or operating a transit, you should remember the following points:

1. The plate bubble follows the direction of the left thumb when you are manipulating the leveling screws.
2. You should always check to see if the plumb bob is still over the point after leveling it. If the plumb bob has shifted, you should recenter the instrument.
3. While loosening the two adjacent leveling screws, you can shift the transit head laterally.

4. You should always maintain contact between the leveling screw shoes and the foot plate.

5. You should not disturb the setup of the instrument until you are certain that all observations at that point are completed and roughly checked. You should move the instrument from that setup only after checking with the party chief.

6. Before the transit is moved or taken up, you should center the instrument on the foot plate, roughly equalize the height of the leveling screws, clamp the upper motion (the lower motion may be tightened lightly), and point the telescope vertically upward and also lightly tighten the vertical motion clamp.

Measuring Horizontal Angles

The transit contains a graduated horizontal circle, referred to as the horizontal limb. The horizontal limb may be graduated clockwise from 0° through 360°, as shown in figure 13-9, view A, or clockwise from 0° through 360° and also in quadrants, as shown in figure 13-9, view B; or both clockwise and counterclockwise from 0° through 360°, as shown in figure 13-9, view C.

The horizontal limb can be clamped to stay fast when the telescope is rotated (called clamping the lower motion), or it can be released for rotating by hand (called releasing the lower motion).

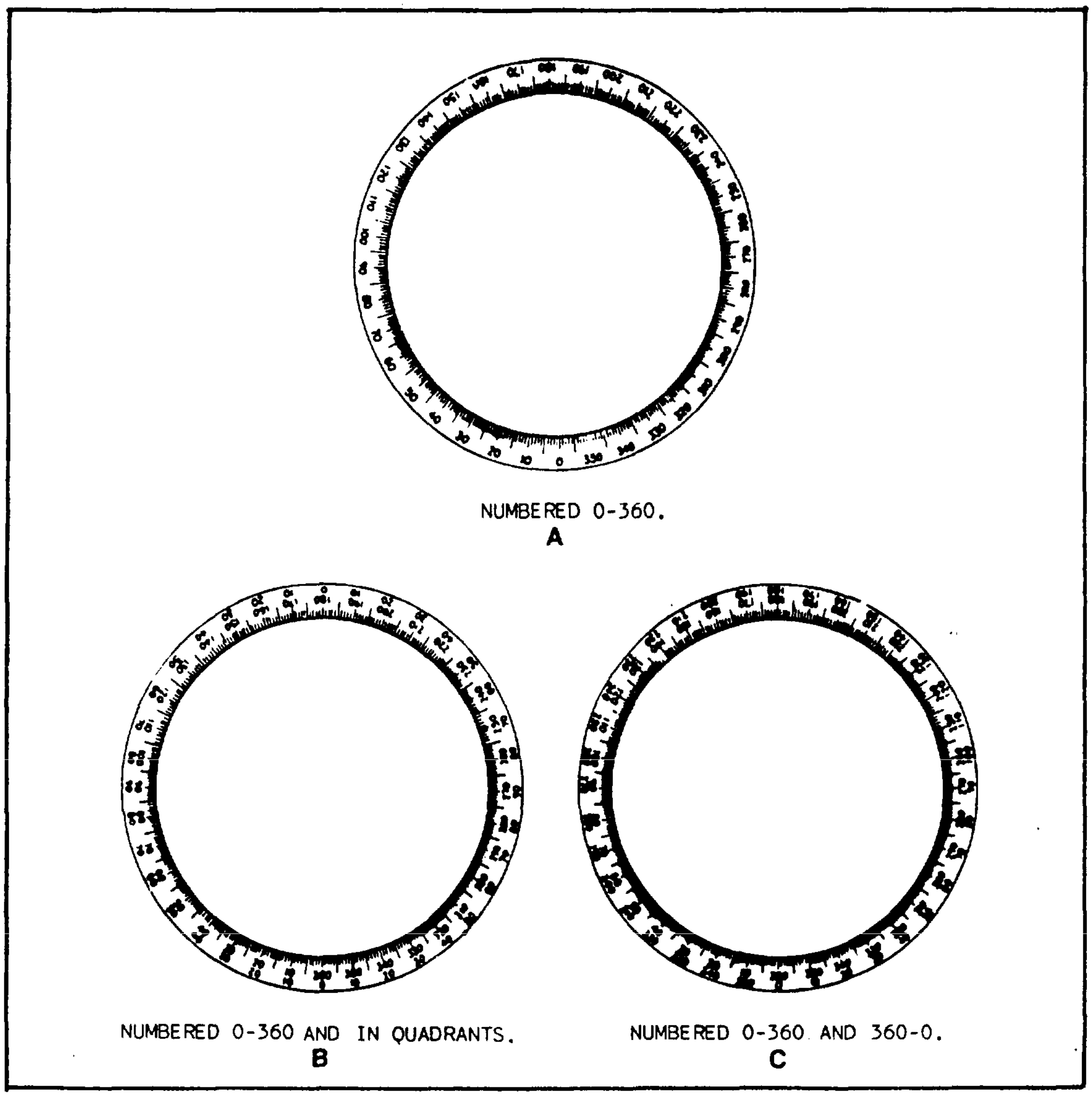


Figure 13-9.-Three types of horizontal limb graduations.

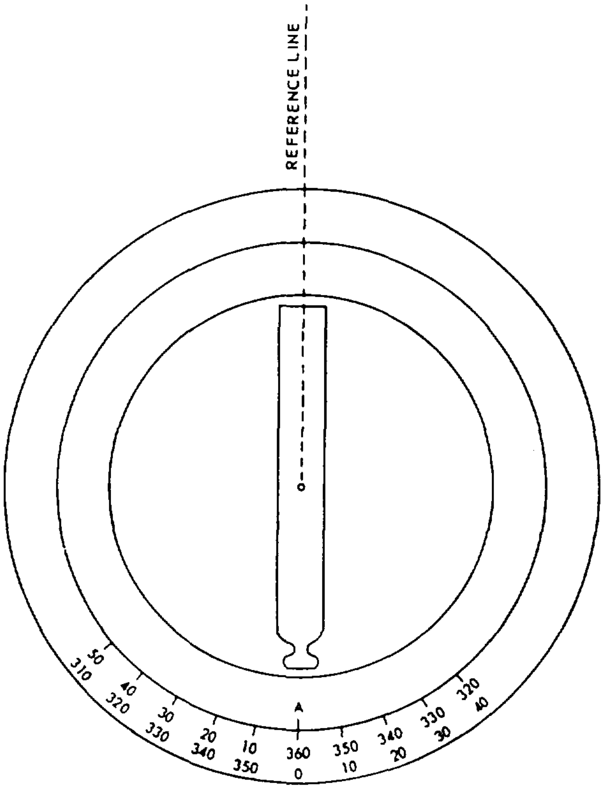


Figure 13-10.-Setting the vernier at zero-zero.

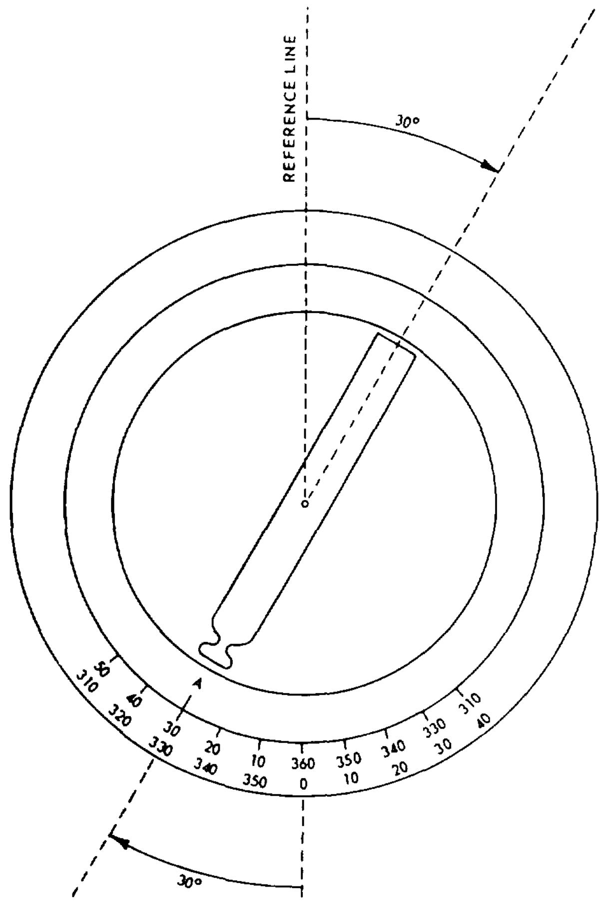


Figure 13-11.-Setting an angle exactly on the vernier zero.

The horizontal limb is paired with another circle (the vernier plate), which is partially graduated on either side of zero graduations located 180° apart on the plate. When the telescope is in the normal (upright) position, the A vernier is located vertically below the eyepiece, and the B vernier, below the objective end of the telescope. The zero on each vernier is the indicator for reading the sizes of horizontal angles turned on the horizontal limb.

Figures 13-10 and 13-11 illustrate the method of turning an angle of 30° from a reference line with a transit.

1. With the transit properly set over the point or station, bring one of the horizontal verniers near zero by hand; clamp the upper motion; and, by turning the upper tangent screw, set one vernier at 0°, usually starting with the A vernier (fig. 13-10). Train the telescope to sight the marker (range pole, chaining pin, or the like) held on the reference line; clamp the lower motion; and, by using the lower tangent screw, set the line of sight on the marker.

2. Release the upper motion and rotate the telescope to bring the zero on the A vernier in line with th3 30° graduation on the horizontal limb, as shown in figure 13-11. To set the vernier exactly at 30°, use the upper tangent screw. You may use a magnifying glass to set the vernier easily and accurately.

3. Mark the next point with a marker, and follow the procedures for establishing a point or station.

Similarly, you may use the procedures above to measure a horizontal angle by sighting on two existing points and reading their interior angle. In addition, the following hints may help you when you are taking horizontal measurements:

1. Make the centering of the line of sight as close as possible by hand so that you will not turn the tangent screw more than one or two turns. Make the last turn of the tangent screw clockwise to compress the opposing springs.

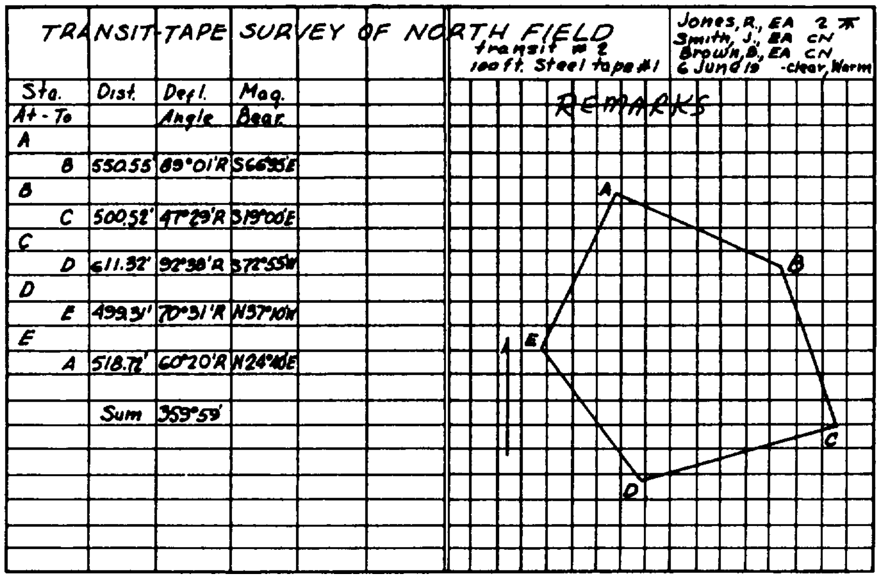


Figure 13-12.-Sample field notes from a deflection angle transit-tape survey.

2. Read the vernier with the eye directly over the top of the coinciding graduations to eliminate the effects of parallax.

3. Take the reading of the other vernier as a check. The readings should be 180° apart.

4. Check the plate bubbles before measuring an angle to see if they are centered, but do not disturb the leveling screws between the initial and final settings of the line of sight. If an angle is measured again, the plate may be releveled after each reading before sighting again on the starting point.

5. Make sure that the rodman is holding the range pole truly vertical when you sight at it. When the bottom of the range pole is not visible, let the rodman use a plumb bob.

6. Avoid accidental movement of the horizontal circle; for instance, moving the wrong clamp or tangent screw. If a number of angles will be observed from one setup without moving the horizontal circle, you should sight at some clearly defined distant object that will serve as a reference mark and take note of the angle. Occasionally, you should recheck the reading to this point during measurement to see if there is any accidental movement.

An example of a horizontal deflection angle measurement is shown in figure 13-12. The field notes contain data taken from a loop traverse shown in the sketch. The transit was first set up at station A, and the magnetic bearing of AB was read on the compass. Then the deflection angle between the extension of EA and AB was turned in the following manner:

1. The instrumentman released both clamps, matched the vernier to zero by hand, tightened the upper motion clamp, and set the zero exactly with the upper tangent screw.

2. With the telescope plunged (inverted position), the instrumentman sighted the range pole held on station E. Then he tightened the lower motion clamp and manipulated the lower motion tangent screw to bring the vertical cross hair to exact alignment with the range pole.

3. The instrumentman replunged the telescope and trained on the extension of EA. (Notice that the telescope is in its normal position now.) He then released the upper motion and rotated the telescope to the right until the vertical cross hair came into line with the range pole held on station B. He further set the upper motion clamp screw and brought the vertical cross hair into exact alignment with the range pole by manipulating the upper motion tangent screw.

4. The instrumentman then read the size of the deflection angle on the A vernier (89°01´). Since the angle was turned to the right, he recorded 89°01´R in the column headed "Defl. Angle." Likewise, he recorded the chained distance between stations A and B and the magnetic bearing of traverse line AB under their appropriate headings.

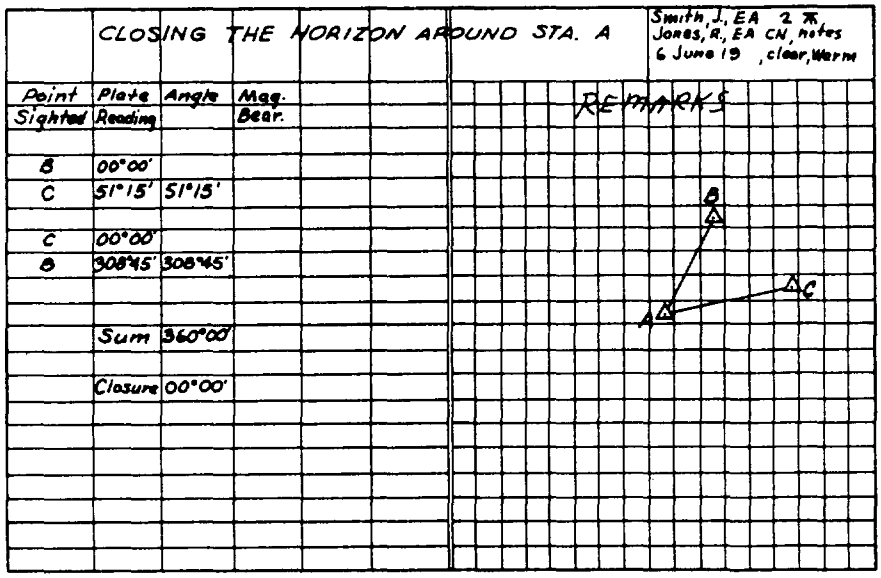


Figure 13-13.-Sample field notes for closing the horizon.

The instrumentman used the same method at each traverse station, working clockwise around the traverse to station E. Note that the algebraic sum of the measured deflection angle (angles to the right considered as plus, to the left as minus) is 350°59´. For a closed traverse, the algebraic sum of the deflection angles from the standpoint of pure geometry is 360°00´. Therefore, there is an ANGULAR ERROR OF CLOSURE here of 0°01´. This small error would probably be considered a normal error. A large variance would indicate a larger mistake made in the measurements.

In the example just presented, the general accuracy of all the angular measurements was checked by comparing the sum of the deflection angles with the theoretical sum. The accuracy of single angular measurement can be checked by the, procedure CLOSING THE HORIZON. The method is based on the fact that the theoretical sum of all the angles around a point is 360°00´.

The field notes in figure 13-13 show the procedure for closing the horizon. The transit was set up at station A, and angle BAC was turned, measuring 51°15´. Then the angle from AC clockwise around to AB was turned, measuring 308°45´. The sum of the two angles is 360°00´. The angular error of closure is therefore 0°00´, meaning that perfect closure is obtained.

Measuring Vertical Angles

The vertical circle and the vertical vernier of a transit were discussed in chapter 11 of this training manual. They are used for measuring vertical angles.

A vertical angle is the angle measured vertically from a horizontal plane of reference. (See fig. 13-14, view A.) When the telescope is pointed in the horizontal plane (level), the value of the vertical angle is zero. When the telescope is pointed up at a higher feature (elevated), the vertical angle increases from zero and is a PLUS VERTICAL ANGLE or ANGLE OF ELEVATION. These values increase from 0° to +90° when the telescope is pointed straight up.

As the telescope is depressed (pointed down), the angle also increases in numerical value, A depressed telescope reading, showing that it is below the horizontal plane, is a MINUS VERTICAL ANGLE or ANGLE OF DEPRESSION. These numerical values increase from 00 to –90° when the telescope is pointed straight down.

To measure vertical angles, you must set the transit upon a definite point and level it. The plate bubbles must be centered carefully, especially for transits that have a fixed vertical vernier. The line of sight is turned approximately at the point; the horizontal axis is clamped. Then, the horizontal cross hair is brought exactly to the point by means of the telescope tangent screw. The angle is read

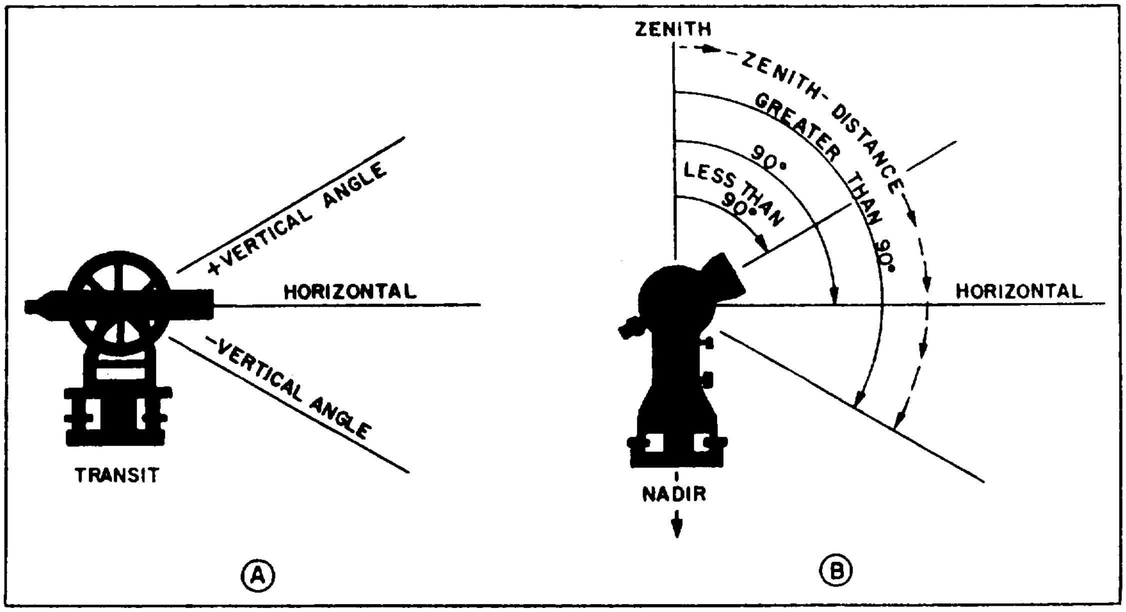


Figure 13-14.-Vertical angles and zenith distances.

on the vertical limb by means of the vertical vernier.

On a transit with a movable vertical vernier, the vernier is equipped with a control level. The telescope is centered on the point as described above, but the vernier bubble is centered before the angle is read.

The ZENITH is an imaginary point overhead where the extension of the plumb line will intersect an assumed sphere on which the stars appear projected. The equivalent point, directly below the zenith, is the NADIR. Use of the zenith permits reading angles in a vertical plane without using a plus or a minus. Theodolites have a vertical scale reading zero when the telescope is pointed at the zenith instead of in a horizontal plane. With the telescope in a direct position and pointed straight up, the reading is 0°; on a horizontal line, the reading is 90°; and straight down, 180°. When measuring vertical angles with the theodolites (fig. 13-14, view B), you should read the angle of elevation with values less than 90°and the angle of depression with values greater than 90°. These angle measurements with the zenith as the zero value are called the ZENITH DISTANCES. DOUBLE ZENITH DISTANCES are observations made with the telescope direct and reversed to eliminate errors caused by the inclination of the vertical axis and the collimation of the vertical circle.

Zenith distance is used in measuring vertical angles involving trigonometric leveling (discussed in the next chapter) and in astronomical observations. (See Engineering Aid 1 & C, NAVEDTRA 10635-C.)

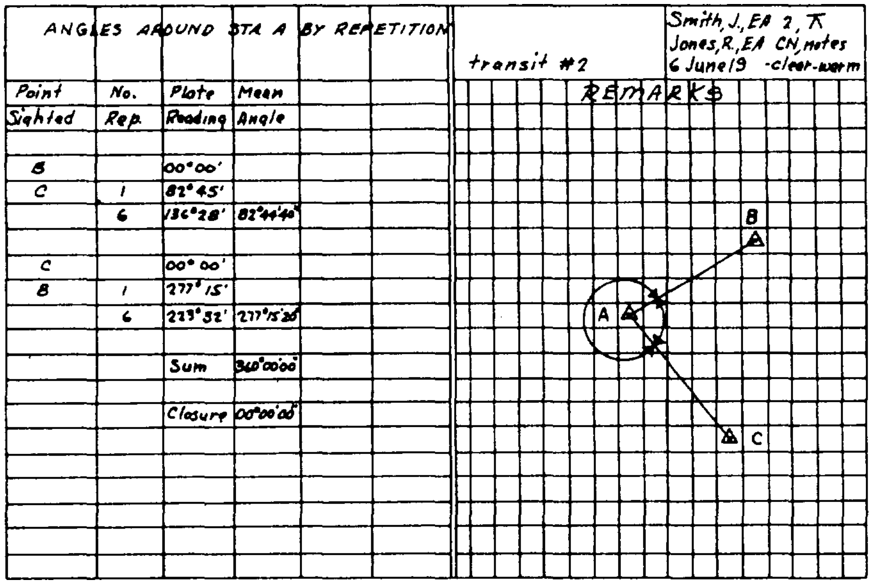
Measuring Angles by Repetition

You may recall, from a previous discussion, the distinction between precision and accuracy. A transit on which angles can be measured to the nearest 20 sec is more precise than one that can measure only to the nearest 1 min. However, this transit is not necessarily more accurate.

The inherent angular precision of a transit can be increased by the process of REPETITION. To illustrate this principle, suppose that with a 1-min transit you turn the angle between two lines in the field and read 45°00´. The inherent error in the transit is 1´; therefore, the true size of this angle is somewhere between 44°59´30´´ and 45°00´30´´.

For example, when using repetition, you leave the upper motion locked but release the lower motion. The horizontal limb will now rotate with the telescope, holding the reading of 45°00´. You plunge the telescope, train again on the initial line of the angle, and again turn the angle. You have now doubled the angle. The A vernier should read approximately 90°00´.

For this second reading, the inherent error in the transit is still 1 min, but the angle indicated



ANGLES AROUND STA A BY REPETITION

Point Sighted	No. Rep.	Plate Reading	Mean Angle
B		00°00′	
C	1	82°45′	
	6	136°28′	82°44′40″
C		00°00′	
B	1	277°15′	
	6	223°32′	277°15′20″
		Sum	360°00′00″
		Closure	00°00′00″

transit #2

Smith, J., EA 2, ⊼
Jones, R., EA CN, notes
6 June 19 -clear-warm

REMARKS

Figure 13-15.-Notes for the angle around a station, repeated six times.

on the A vernier is about twice the size of the actual angle measured. The effect of this is to halve the total possible error. This error was originally plus or minus 30 sec. Now, the error is plus or minus only 15 sec.

If you measure this angle a total of six times, the total possible error will be reduced to one-sixth of 30 see, or plus or minus 5 sec. In theory, you could go on repeating the angle and increasing the precision indefinitely. In actual practice, because of lost motion in the instrument and accidental errors, it is not necessary to repeat the angle more than six times.

The observation may be taken alternately with the telescope plunged before each subsequent observation. But a much simpler way is to take the first half of the observations with the telescope in the normal position, the other half, in an inverted position. In the example given above, the first three readings may be taken when the telescope is in its normal position; the last three when it is in its reversed position. To avoid the effect of tripod twist, after each repetition, rotate the instrument on its lower motion in the same direction that it was turned during the measurement; that is, the direction of movement should always be either clockwise or counterclockwise.

Measuring angles by repetition eliminates certain possible instrumental errors, such as those caused by eccentricity and by nonadjustment of the horizontal axis.

Figure 13-15 shows field notes for the angle around a station, repeated six times. The angle BAC was measured six times, and the angle closing the horizon around station A was also measured six times. The first measurement is not a true repeat, but it is counted as one in the column headed "No. Rep." (number of repetitions).

With the transit first trained on B and the zeros matched, the plate reading was 00°00′. This is recorded beside B in the column headed "Plate Reading." The upper motion clamp was then released, the telescope was trained on C, and a plate reading of 82°45′ was obtained. This reading is recorded next to the figure "1" (for "1st repetition") in the column headed "No. Rep." The measurement of angle BAC was then repeated five more times. After the final measurement, the plate reading was 136°28′. This plate reading is recorded as the sixth repetition.

Now to get the mean angle, it is obvious that you need to divide some number, or figure, by the total number of repetitions. The question is, what figure? To determine this, you first multiply the initial measurement by the total number of repetitions. In this case, this would be as follows:

$$82°45' \times 6 = 496°30'.$$

Next, you determine the largest multiple of 360° that can be subtracted from the above product. Obviously, the only multiple of 360° that can be subtracted from 496°30′ is 360°. This multiple is then added to the final measurement to obtain the figure that is to be divided by the total number of repetitions. In this example,

$$136°28' + 360° = 496°28'.$$

The mean angle then is

$$496°28' \div 6 = 82°44'40''.$$

Enter this in the column headed "Mean Angle."

The following computation shows that you should use the same method to obtain the mean closing angle.

$$277°15' \times 6 = 1663°30'$$

$$360° \times 4 = 1440° \text{ (largest multiple of } 360°)$$

$$223°32' + 1440° = 1663°32'$$

$$1663°32' \div 6 = 277°15'20''$$

In the example shown above, the sum of the mean angle (82°44´40´´) and the mean closing angle (277°15´20´´) equals 360°00´00´´. This reflects perfect closure. In actual practice, perfect angle closure would be unlikely.

RUNNING A DISTANCE (LINE)

It is often necessary to extend a straight line marked by two points on the ground. One of the methods discussed below may be used depending on whether or not there are obstacles in the line ahead, and, if so, whether the obstacles are small or large.

Double Centering or Double Reversing

This method is used to prolong or extend a line. Suppose you are extending line AB, shown in figure 13-16. You set up the transit at B, backsight on A, plunge the telescope to sight ahead, and set the marker at C´. With the telescope still inverted, you again sight back on A; but this time do it by rotating the telescope through 180°. You then replunge the telescope and mark the point C´´. Mark the point C halfway between C´ and C´´. This is the point on the line

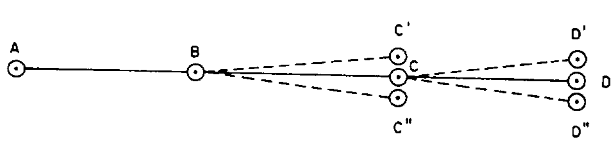


Figure 13-16.-Double centering.

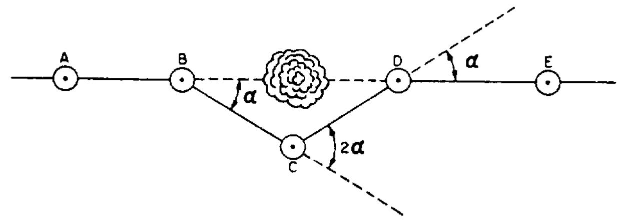


Figure 13-17.-Bypassing a small obstacle by the angle offset method.

AB you need to extend. If the instrument is in perfect adjustment (which seldom happens), points C´ and C´´ will coincide with point C. For further extension, the instrument is moved to C and the procedure repeated to obtain D.

Bypassing an Object by Angle Offset

This method is applied when a tree or other small obstacle is in the line of sight between two points. The transit or theodolite is set up at point B (fig. 13-17) as far from the obstacle as practical. Point C is set off the line near the obstacle and where the line BC will clear the obstacle. At B, measure the deflection angle a. Move the instrument to C, and lay off the deflection angle 2a. Measure the distance BC, and lay off the distance CD equal to BC. Move the instrument to D, and lay off the deflection angle α. Mark the point E. Then, line DE is the prolongation of the line AB.

Bypassing an Object by Perpendicular Offset

This method is used when a large obstruction, such as a building, is in the line of sight between two points. The solution establishes a line parallel to the original line at a distance clear from the obstacle, as shown in figure 13-18. The instrument

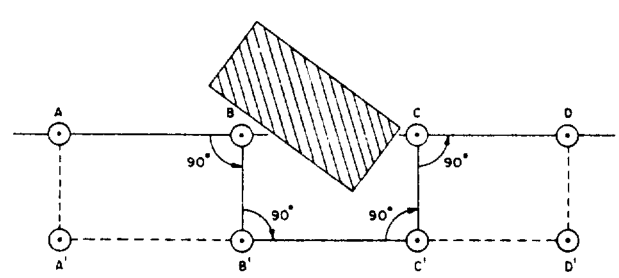


Figure 13-18.-Bypassing a large obstacle by the perpendicular offset method.

is set up at B, and a 90° angle is turned from line AB. The distance BB´ is carefully measured and recorded. The instrument is moved to B´, and another 90° angle is turned. B´C´ is laid off to clear the obstacle. The instrument is moved to C, and a third 90° angle is turned. Distance CC´, equal to BB´, is measured and marked. This establishes a point C on the original line. The instrument is moved to C, and a fourth 90° angle is turned to establish the alignment CD that is the extension of AB beyond the obstacle.

When the distance to clear the obstacle, BB´ or CC´, is less than a tape length, you can avoid turning four 90° angles as follows: Erect perpendicular offsets from points A and B in figure 13-18 so that AA´ equals BB´. Set up the instrument at B', and measure angle A´B´B to be sure that it's 90°. Extend line A´B´ to C´ and then to D´, making sure that point C clears the obstacle. Then, lay off perpendicular offset C´C equal to AA´ or BB´ and perpendicular offset D´D equal to C´C. Then, line CD is the extension of line AB. The total distance of the line AD is the sum of the distances AB, B´C´, and CD.

You also compute the diagonals formed by the end rectangles and compare the result to the actual measurement, if you can, as a further check.

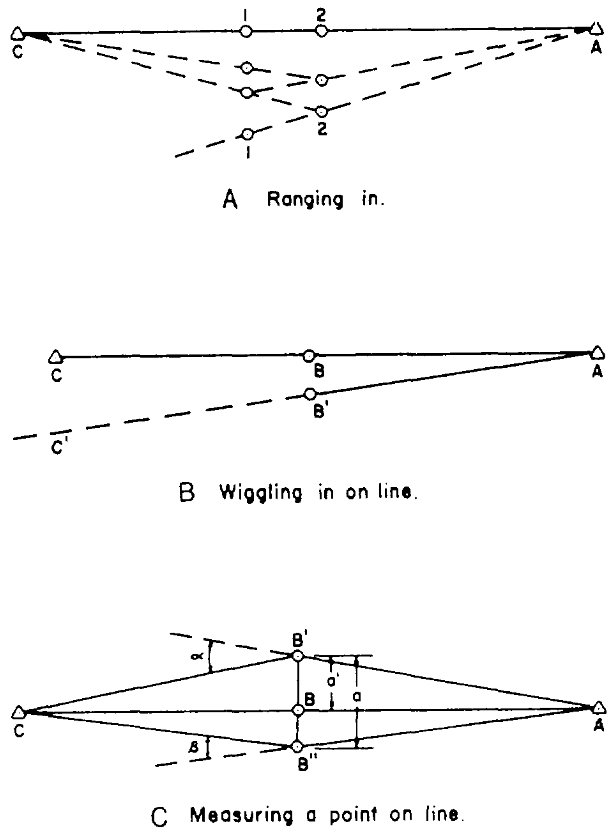


Figure 13-19.-Setting up on a line between two points.

Line Between Nonintervisible Points

Sometimes you need to run a straight line between nonintervisible points when events make the use of the above methods of bypassing an obstacle impractical. If there is an intermediate point on the straight line from which both of the end points can be observed, the method called BALANCING IN (also called BUCKING IN, JIGGLING IN, WIGGLING IN, or RANGING IN) may be used.

A problem often encountered in surveying is to find a point exactly on the line between two other points when neither can be occupied or when an obstruction, such as a hill lies between the two points. The point to be occupied must be located so that both of the other points are visible from it. The process of establishing the intermediate point is known as wiggling in or ranging in.

The approximate position of the line between the two points at the instrument station is first estimated by using two range poles. The range poles are lined in alternately in the following manner. In figure 13-19, view A, set range pole 1 and move range pole 2 until it is exactly on line between pole 1 and point A. You do this by sighting along the edge of pole 1 at the station *A* until pole 2 seems to be on line. Set range pole 2 and move pole 1 until it is on line between pole 2 and point C. Now, move pole 2 into line again, then pole 1, alternately, until both are on line AC. The line will appear to pass through both poles and both stations from either viewing position.

After finding the approximate position of the line between the two points, you set up the instrument on this line. The instrument probably will not be exactly on line, but will be over a point, such as *B´*, (see fig. 13-19, view B). With the instrument at *B´*, you backsight on A and plunge the telescope and notice where the line of sight *C* passes the point C. Estimate this distance CC´ and also the distance that *B´* would be away from C and A. Estimate the amount to move the instrument to place it on the line you need. Thus, if *B´* is midway between *A* and C, and C´ misses *C* by 3 feet to the left, *B´* must be moved about 1.5 feet to the right to reach *B*. Continue the sequence of backlighting, plunging the telescope, and moving the instrument until the line of sight

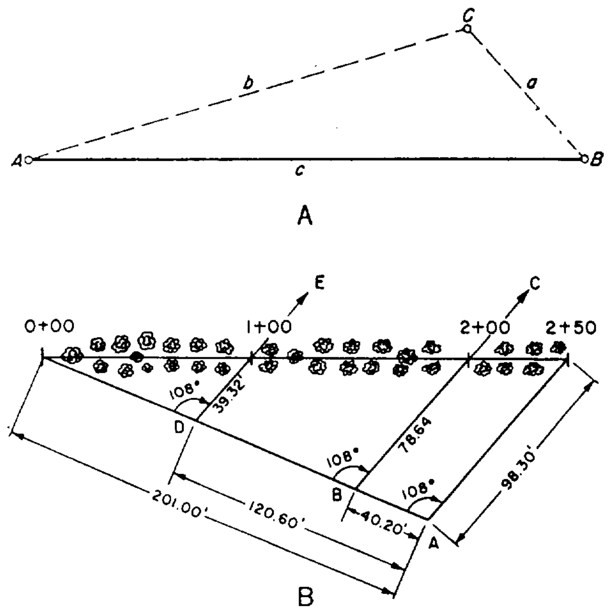


Figure 13-20.-Random line method of locating intermediate stations.

passes through both *A* and *C*. When you do this, the telescope is reversed, but the instrument is not rotated. This means that if the telescope is reversed for backlighting on *A*, all sightings on *A* are made with the telescope reversed. Mark a point on the ground directly under the instrument. Then, you continue to use this method with the telescope direct for each backsight on *A*. Mark a second point on the ground. The point you need on the line *AC* is then the midpoint between the two marked points.

The method outlined above is usually time consuming. Even though the shifting head of the instrument is used in the final instrument movements, you may have to pick up and move the instrument several times. The following method often saves time. After finding the approximate position of the line between the two points, you mark two points *B´* and *B´´*, (fig. 13-19, view C), 1 or 2 feet apart where you know they straddle the line *AC*. Set up over each of these two points in turn and measure the deflection angles α and β. Also measure the horizontal distance a, between points *B´* and B´´. Then you can find the position *B* on the line *AC* by using the following equation:

$$a' = a \frac{\alpha}{\alpha + \beta}$$

in which a' is the proportionate offset distance from *B´* toward B´´ for the required point *B*, and α and β are both expressed in minutes or in seconds.

RANDOM LINE

It is sometimes necessary to run a straight line from one point to another point that is not visible from the first point. If there is an intermediate point on the line from which both end-points are visible, this can be done by the balancing-in method described previously. If no such intermediate point exists, the RANDOM LINE method illustrated in figure 13-20, view A may be used.

The problem here is to run a line from A to B, B being a point not visible from A. It happens, however, that there is a clear area to the left of the line AB, through which a random line can be run to C; C being a point visible from A and B.

To train a transit set up at A on B, you must know the size of the angle at A, You can measure side b and side a, and you can measure the angle at C. Therefore, you have a triangle in which you know two sides and the included angle. You can solve this triangle for angle A by (1) determining the size of side c by the law of cosines, then determining the size of angle A by the law of sines, (2) solving for angle A by reducing to two right triangles.

Suppose you find that angle A measures 16°35´. To train a transit at A on B, you would simply train on C and then turn 16°35´ to the right.

You may also use the random line method to locate intermediate stations on a survey line, In figure 13-20, view B, stations 0 + 00 and 2 + 50, now separated by a grove of trees, were placed at some time in the past. You need to locate stations 1 + 00 and 2 + 00, which lie among the trees.

Run a line at random from station 0 + 00 until you can see station 2 + 50 from some point, A, on the line. The transitman measures the angle at A and finds it to be 108°00´. The distances from A to stations 0 + 00 and 2 + 50 are chained and found to be 201.00 ft and 98.30 ft. With this information, it is now possible to locate the intermediate stations between stations

0 + 00 and 2 + 50. The distances AB and AD can be computed by ratio and proportion, as follows:

$$AB = \frac{50}{250} \times 201.0 = 40.20 \text{ ft}$$

$$\text{and } AB = \frac{150}{250} \times 201.0 = 120.60 \text{ ft.}$$

These distances are laid off on the random line from point A toward station 0 + 00. The instrumentman then occupies points B and D; turns the same angle, 108°00′, that he measured at point A; and establishes points C and E on lines from points B and D through the stations being sought. The dist antes are computed by similar triangles as follows:

$$\text{B to station } 2 + 00(BC) = \frac{200}{250} \times 98.3 \text{ ft} = 78.64 \text{ ft}$$

$$\text{D to station } 1 + 00(DE) = \frac{100}{250} \times 98.3 \text{ ft} = 39.32 \text{ ft}$$

TYING IN A POINT

Determining the horizontal location of a point or points with reference to a station, or two stations, on a traverse line is commonly termed *TYING IN THE POINT.* Various methods used in the process are presented in the next several paragraphs.

Locating Points by Swing Offsets

The SWING OFFSET is used for locating points close to the control lines (fig. 13-21). Measurement of a swing offset distance provides an accurate determination of the perpendicular distance from the control line to the point being located. The swing offset is somewhat similar to the range tie (explained later), but as a rule, requires no angle measurement. To determine the swing offset distance, a chainman holds the zero mark of the tape at a corner of the structure while another chainman swings an arc with the graduated end of the tape at the transit line AB. When the shortest reading on the graduated end of the tape is observed, the swing offset or perpendicular distance to the control line is obtained at points a or b. In addition, the horizontal distances between the instrument stations (A and B) and the swing offset points (a and b) maybe measured and marked. A tie distance and angle α or Φ may be measured from either instrument station to the corner of the structure to serve as a check.

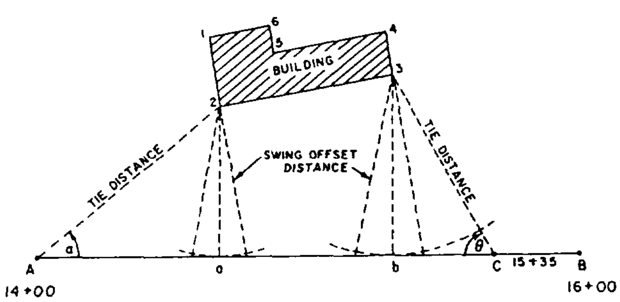


Figure 13-21.-Swing offset method of locating points.

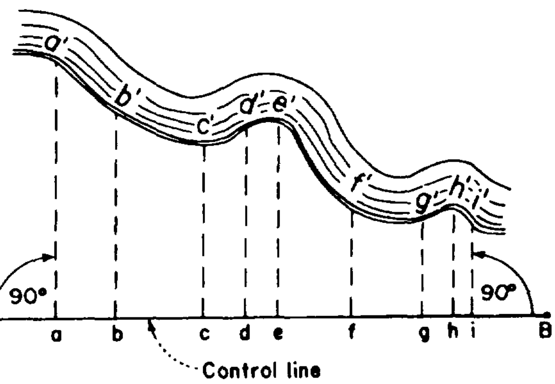


Figure 13-22.-Perpendicular offsets.

Locating Points by Perpendicular Offsets

The method of PERPENDICULAR OFFSETS from a control line (fig. 13-22) is similar to swing offsets. This method is more suitable than the swing offset method for locating details of irregular objects, such as stream banks and winding roads. The control line is established close to the irregular line to be located, and perpendicular offsets, aa′, bb′, cc′, and so on, are measured to define the irregular shape. When the offset distances are short, the 90° angles are usually estimated; but when the distances are several hundred feet long, the angles should be laid off with an instrument. The distances to the offset points from a to i are measured along the control line.

Locating Points by Range Ties

A point's location can also be determined by means of a RANGE TIE, using an angle and a

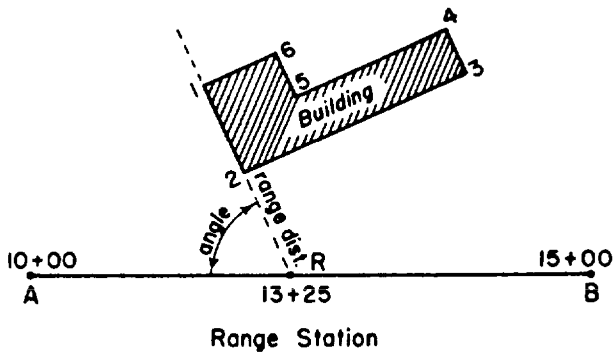


Figure 13-23.-Range ties.

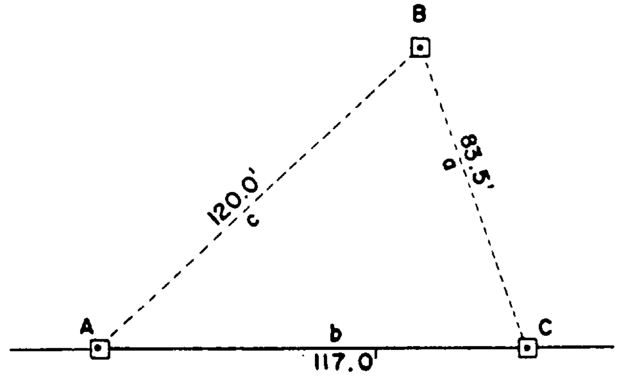


Figure 13-24.-Locating a point by distances from two stations.

distance. The method requires extra instrument manipulation and should be used only when none of the previous methods are satisfactory for use. Actually, range ties establish not only the corner of a structure but also the alignment of one of the sides. In figure 13-23, assume that the building is not visible from either A or B or that either or both of the distances from A to B to a corner of the building cannot be measured easily. With the instrument set up at either A or B and the line AB established, one member of the party moves along AB until he reaches point R, which is the intersection of line 1-2 extended. The instrument is moved and set up on R, and the distance along the line AB to R is measured. An angle measurement to the building is made by using either A or B as the backsight. The range distance, R-2, is measured as well as the building dimensions.

SETTING ADJACENT POINTS

"To set a point adjacent to a traverse line" means to establish the location of a point by following given tie data. This tie data may be (1) a perpendicular offset distance from a single specified station, (2) angles from two stations, or (3) an angle from one station and the distance from another station.

Setting Points When Given a Perpendicular Offset Distance

To set a point when given an angle and its distance from a single station, you simply setup the instrument at the station, turn the designated angle, and chain the distance along the line of sight. For perpendicular offset, the angle is 90°.

To set a point when given a distance from each of two stations, you can manage by using two tapes if each of the distances is less than a full tape length. To do so, you set the zero end of the tapes on both stations, run out the tapes, and match the distance mark on each tape to correspond with the required distance from the stations. When the tape is drawn taut, the point of contact between the tapes will be over the location of the desired point.

If one or both of the distances is greater than a full tape length, you can determine direction of one of the tie lines by correct triangle solution. For example, in figure 13-24, you want to set Point B 120.0 ft from station A and 83.5 ft from station C. A and C are 117.0 ft apart. You can determine the size of the angle at A by triangle solution as follows:

$$1 - \cos A = \frac{2(s - b)(s - c)}{bc}$$

$$s = 1/2(120.0 + 117.0 + 83.5) = 160.25$$

$$1 - \cos A = \frac{2(43.25)(40.25)}{(117.0)(120.0)} = 0.24797$$

$$\cos A = 1.00000 - 0.24797 - 0.75203$$

$$A = 41°14'$$

To set point B, you can set up a transit at A, sight on C, turn 41°14′ to the left, and measure off 120.0 ft on that line of sight. As a check, you can measure BC to be sure it measures 83.5 ft.

Setting Points When Given Angles from Two Stations

To set a point when given the angle from each of two traverse stations, you should ordinarily

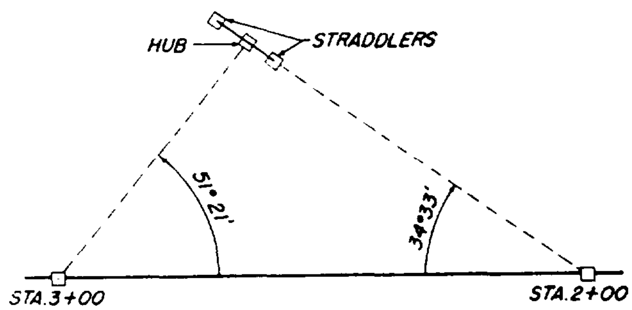


Figure 13-25.-Setting a point by the use of straddlers.

use a pair of straddle hubs (commonly called STRADDLERS), as shown in figure 13-25. Here the point was to be located at an angle of 34°33´ from station 2 + 00 and at an angle of 51°21´ from station 3 + 00. The transit was set up at station 2 + 00, sighted on station 3 + 00, and an angle of 34°33´ was turned to the right. On this line of sight, a pair of straddle hubs was driven, one on either side of the estimated point of intersection of the tie lines. A cord was stretched between the straddlers.

The transit was then shifted to station 3 + 00, sighted on station 2 + 00, and an angle of 51°21´ was turned to the left. A hub was driven at the point where this line of sight intercepted the cord between the straddlers.

Setting Points When Given an Angle from One Station and the Distance from Another

To set a point with a given angle from one station and the distance from another, you would find it best to determine the direction of the distance line by triangle solution. In figure 13-26,

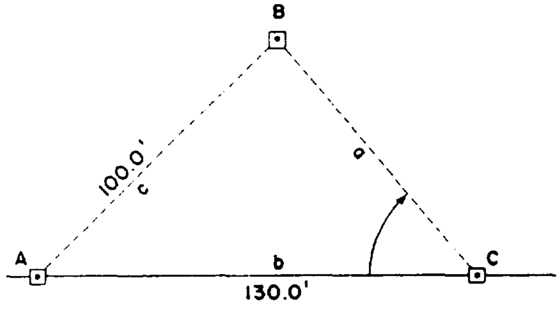


Figure 13-26.-Locating a point by angle and distance from two stations.

point B is to be located 100.0 ft from station A and at an angle of 50°00´ from station C.

In this example, you can determine the size of the angle at A by first determining the size of angle B, then subtracting the sum of angles B and C from 180°. The solution for angle B is as follows:

$$\sin B = \frac{130.0 \sin 50°00'}{100.0}$$

$$\sin B = \frac{130.0(0.76604)}{100.0} = 0.99585.$$

Angle B then measures, to the nearest minute, 84°47´. Therefore, angle A measures

$$180°00' - (84°47' + 50°00') = 45°13'.$$

Set up a transit at A, sight on C, and turn 45°13´ to the left. Then, you set B by measuring off 100.0 ft on this line of sight. As a check, you set up the transit at C, sight on A, turn 50°00´ to the right, and make sure this line of sight intercepts the marker at B.

TRANSIT-TAPE SURVEY

The exact method used in transit-tape survey may vary slightly, depending upon the nature of the survey, the intended purpose, the command or unit policy, and the preferences of the survey party chief. The procedures presented in this section are customary methods described in general terms.

SELECTING POINTS FOR MARKING

All points where a traverse changes direction are marked, usually with a hub that locates the station exactly, plus a guard stake on which the station of the change-of-direction point is inscribed, such as 12 + 35. In the expression “station 12 + 35,” the 12 is called the full station and the 35 is called the plus.

The points that are to be tied to the traverse or set in the vicinity of the traverse are usually selected and marked or set as the traverse is run. The corresponding tie stations on the traverse are selected and marked at the same time. The first consideration in selecting tie stations is VISIBILITY, meaning that tie stations and the point to be tied or set must be intervisible. The next is PERMANENCY (not easily disturbed). Last is the STRENGTH OF INTERSECTION, which generally means that the angle between two tie lines should be as close to 90° as possible. The more acute or obtuse the angle is between tie lines, the less accurate the location of the point defined by their intersection.

IDENTIFYING PARTY PERSONNEL

A typical transit-tape survey party contains two chainmen, a transitman, a recorder (sometimes the transitman or party chief doubles as recorder), a party chief (who may serve as either instrumentman or recorder, or both), and axmen, if needed. The transitman carries, sets up, and operates the transit; the chainmen do the same with the tapes and the marking equipment.

When the transitman turns an angle, he calls out the identity and size of the angle to the recorder, as "Deflection angle AB to BC, 75°16′, right." The recorder repeats this, then makes the entry. Similarly, the head chainman calls out the identity and size of a linear distance, as "B to C, 265.72 ft," then the recorder repeats this back and makes the entry at that time. If the transitman closes the horizon around a point, he calls out, "Closing angle, such and such." The recorder repeats this and then adds the closing angle to the original angle. If the sum of the angles doesn't come close to 360°, the recorder notifies the party chief.

The party chief is in complete charge of the party and makes all the significant decisions, such as the stations to be marked on the traverse.

ATTAINING THE PRESCRIBED ORDER OF PRECISION

The important distinction between accuracy and precision in surveying is explained as follows:

- Accuracy denotes the degree of conformity with a standard. It relates to the quality of a result and is distinguished from precision, which relates to the quality of the operation by which the result is obtained.

The accuracy attained by field surveys is the product of the instructions or specifications to be followed in doing the work and the precision in following those instructions.

For example, the "accuracy of a surveyor's tape" means the degree to which an interval of 100 ft, as measured on the tape, actually agrees with the exact interval of a standard 100-ft tape. If a tape indicates 100 ft when the interval it measures is only 99.97 ft, the tape contains an inaccuracy of 0.03 ft for every 100 ft measured. The accuracy of this particular tape, expressed as a fraction, is 0.03/100, or approximately 1/3,300.

- Precision denotes degree of refinement in the performance of an operation or in the statement of a result. It relates to the quality of execution and is distinguished from accuracy that relates to the quality of the result. The term *precision* not only applies to the fidelity of performing the necessary operations but, by custom, has been applied to methods and instruments used in obtaining results of a high order of accuracy. Precision is exemplified by the number of decimal places to which a computation is carried and a result stated. In a general way, the accuracy of a result should determine the precision of its expression. Precision will not have significance unless accuracy is also obtained.

If you measure a linear distance with a tape graduated in feet that are subdivided into tenths, you can read (without estimation) only to the nearest tenth (0.1) of a foot. But with a tape graduated to hundredths of a foot, you can directly read distances measured to the nearest hundredth (0.01) of a foot. The apparent nearness of the second tape will be greater; that is, the second tape will have a higher precision.

Completely precise measurement is impossible in the nature of things. There is always a built-in or inherent error, amounting to the size of the smallest graduation. Precision for the first tape above, expressed as a fraction, is 0.1/100 or 1/1,000 and for the second tape, 1/10,000.

Precision in measurements is usually expressed in a fractional form with unity as the numerator, indicating the allowable error within a certain limit as indicated by the denominator, such as 1/500. In this case, you are allowed a maximum error of 1 unit per 500 units measured. If your unit of measure is in feet, you are allowed 1 ft for every 500 ft.

In general, any survey has to be carried out accurately, meaning that errors and mistakes have to be avoided. The precision of a survey, however, depends upon the order of precision that is either specified or is implied from the nature of the survey.

The various orders of precision are absolute, not relative, in meaning. Federal agencies control surveys. They are generally classified into four orders of precision; namely, FIRST ORDER, SECOND ORDER, THIRD ORDER, and

Table 13-1.-Control Traverse Order of Precision

ORDER OF PRECISION	MAXIMUM NUMBER OF AZIMUTH COURSES BETWEEN AZIMUTH CHECKS	DISTANCE MEASUREMENT ACCURATE WITHIN	MAXIMUM LINEAR ERROR OF CLOSURE	MAXIMUM ERRORS OF ANGLES
FIRST ORDER	15	$\frac{1}{35,000}$	$\frac{1}{25,000}$	* 2 sec $\sqrt{N}$ or 1.0 sec per station.
SECOND ORDER	25	$\frac{1}{15,000}$	$\frac{1}{10,000}$	* 10 sec $\sqrt{N}$ or 3.0 sec per station
THIRD ORDER	50	$\frac{1}{7,500}$	$\frac{1}{5,000}$	* 30 sec $\sqrt{N}$ or 8.0 sec per station
FOURTH ORDER	--	$\frac{1}{3,000}$	$\frac{1}{1,000}$	2 min or compass

N = the number of stations carrying azimuth.
* Use whichever is smaller in value.

FOURTH ORDER control surveys. The FIRST ORDER is the highest and the FOURTH ORDER, the lowest standard of accuracy.

Because of the type of instruments available in the SEABEEs, most of your surveys may not require a precision higher than a third order survey. When the order of precision is not specified, you may use table 13-1 in this training manual (TM) as a standard for a horizontal control survey when using the traverse control method. For surveys that call for a higher order of precision, you will have to use theodolites to obtain the required precision.

The triangulation control method is discussed fully in *Engineering Aid 1 & C,* NAVEDTRA 10635-C. At present, however, you may have survey problems that require the use of the triangulation method. In such a case, you may use table 13-2 in this TM as a guide for the order of precision if it is not specified in the survey.

The practical significance of a prescribed or implied order of precision lies in the fact that the instruments and methods used must be capable of attaining the required precision. The precision of an instrument is indicated by a fraction in which the numerator is the inherent error. (In a 1-min transit, the inherent error is 1 min.)

The denominator is the total number of units in which the error occurs. For a transit, this last is 90°, or 5,400′. The precision of a 1-min transit. then, is 1/5,400, adequate for a third order survey.

Precision of a tape is given in terms of the inherent error per 100 ft. A tape that can be read to the nearest 0.01 ft has a precision of 0.01/100, or 1/10,000—adequate for second order work.

Attaining Precision with a Linear Error of Closure

For a closed traverse, you should attain a RATIO OF LINEAR ERROR OF CLOSURE that corresponds to the order of precision prescribed or implied for the traverse. The ratio of linear error of closure is a fraction in which the numerator is the linear error of closure and the denominator is the total length of the traverse.

To understand the concept of linear error of closure, you should study the closed traverse shown in figure 13-27. Beginning at station C, this traverse runs N30°E300 ft, thence S30°E300 ft; thence S90°W 300 ft. The end of the closing traverse, BC, lies exactly on the point of beginning, C. This indicates that all angles were turned and all distances chained with perfect accuracy, resulting in perfect closure, or an error of closure of zero feet.

However, in reality, perfect accuracy in measurement seldom occurs. In actual practice,

Table 13-2.-Triangulation Order of Precision

PRECISION	APPLICATION	BASE LINE MEASUREMENT MAX. PROBABLE ERROR	TRIANGLE CLOSURE: MAX. AVERAGE ERROR	LENGTH CLOSURE: MAX. DISCREPANCY BET. MEASURED AND COMPUTED LENGTH BASE LINE
FIRST ORDER	CASE I For city and scientific study survey.	$\frac{1}{1,000,000}$	1.0 sec	$\frac{1}{100,000}$
	CASE II Basic network of U.S.	$\frac{1}{1,000,000}$	1.0 sec	$\frac{1}{50,000}$
	CASE III All other purposes.	$\frac{1}{1,000,000}$	1.0 sec	$\frac{1}{25,000}$
SECOND ORDER	CASE I Area networks and supplemental cross arcs in national net.	$\frac{1}{1,000,000}$	1.5 sec	$\frac{1}{20,000}$
	CASE II Coastal areas, inland waterways, and engineering surveys.	$\frac{1}{500,000}$	3.0 sec	$\frac{1}{10,000}$
THIRD ORDER	Topographic Mapping	$\frac{1}{250,000}$	5.0 sec	$\frac{1}{5,000}$

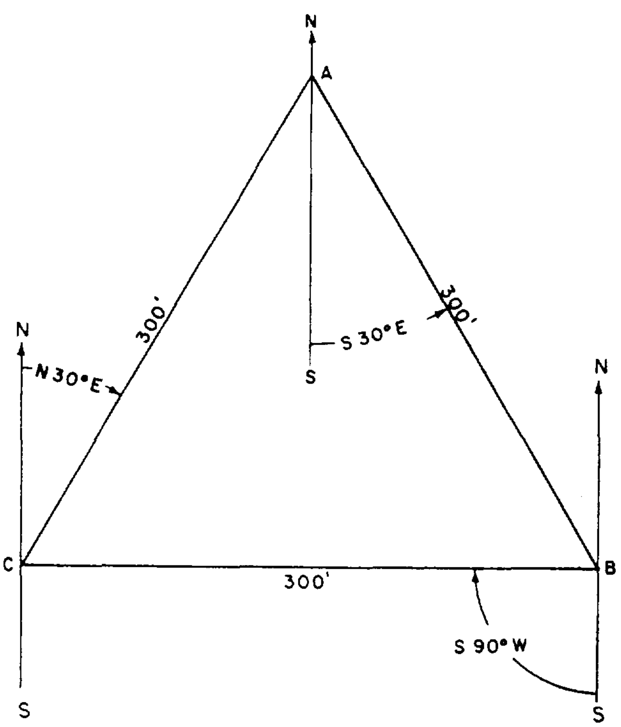


Figure 13-27.-An example of a closed traverse with a perfect (zero-error) closure.

the closing traverse line, BC, shown in figure 13-27, is likely to be some distance from the starting point, C. If this should happen, and, say, the total accumulated linear distance measured along the traverse lines is 900.09 ft, the ratio of error of closure then is .09/900 or 1/10,000. This resulting ratio is equivalent to the precision prescribed for second order work.

Attaining Precision with a Maximum Angular Error of Closure

You know that the sum of the interior angles of a closed traverse should theoretically equal the product of 180° (n – 2), n being the number of sides in the polygon described by the traverse. A prescribed MAXIMUM ANGULAR ERROR OF CLOSURE is stated in terms of the product of a given angular value times the square root of the number of interior angles in the traverse.

Again, if we use the traverse shown in figure 13-27 as an example, the prescribed maximum angular error of closure in minutes is 01 $\sqrt{3}$ because the figure has three interior angles. The sum of the interior angles should be 180°. If the sum of the angles as actually measured and recorded is 179°57′, the angular error of closure is 03′. The maximum permissible error of closure is the product of 01´ times the square root of 3, or about 1.73´. The prescribed maximum angular error of closure has therefore been exceeded.

Meeting Precision Specifications

The following specifications are intended to give you only a general idea of the typical precision requirements for various types of transit-tape surveys. When linear and angular errors of closure are specified, it is understood that a closed traverse is involved.

For many types of preliminary surveys and for land surveys, typical precision specifications may read as follows:

- Transit angles to nearest minute, measured once. Sights on range poles plumbed by eye. Tape leveled by eye, and standard tension estimated. No temperature or sag corrections. Slopes under 3 percent disregarded. Slopes over 3 percent measured by breaking chain or by chaining slope distance and applying calculated correction. Maximum angular error of closure in minutes is $1.5\sqrt{n}$. Maximum ratio linear error of closure, 1/1000. Pins or stakes set to nearest 0.1 ft.

For most land surveys and highway location surveys, typical precision specifications may read as follows:

- Transit angles to nearest minute, measured once. Sights on range poles, plumbed carefully. Tape leveled by hand level, with standard tension by tensionometer or sag correction applied. Temperature correction applied if air temperature more than 15° different from standard (68°F). Slopes under 2 percent disregarded. Slopes over 2 percent measured by breaking chain or by applying approximate slope correction to slope distance. Pins or stakes set to nearest 0.05 ft. Maximum angular error of closure in minutes is $1\sqrt{n}$. Maximum ratio linear error of closure, 1/3,000.

For important boundary surveys and extensive topographical surveys, typical precision specifications may read as follows:

- Transit angles by 1-rein transit, repeated four times. Sights taken on plumb lines or on range poles carefully plumbed. Temperature and slope corrections applied; tape leveled by level. Pins set to nearest 0.05 ft. Maximum angular error of closure in minutes is $0.5\sqrt{n}$. Maximum ratio linear error of closure is 1/5,000.

Note that in the first two specifications, one-time angular measurement is considered sufficiently precise. Many surveyors, however, use two-line angular measurement customarily to maintain a constant check on mistakes.

Measuring Angles vs. Measuring Distances

It is usually the case on a transit-tape survey that the equipment for measuring angles is considerably more precise than the equipment for measuring linear distances. This fact leads many surveyors into a tendency to measure angles with great precision, while overlooking important errors in linear distance measurements.

Making the precision of angular measurement greater than that of linear measurement is useless because your angles are only as good as your linear distances. Suppose that you are running traverse line BC at a right deflection angle of 63°45´ from AB, 180.00 ft to station C. You set up at B, orient the telescope to AB extended, and turn exactly 63°45´00´´ to the right. But instead of measuring off 180.00 ft, you measure off 179.96 ft. Regardless of how precisely you turn all of the other angles in the traverse, every station will be dislocated because of the error in the linear measurement of BC.

Remember that angles and linear distances must be measured with the same precision.

IDENTIFYING ERRORS AND MISTAKES IN TRANSIT WORK

In transit work, errors are grouped into three general categories; namely, INSTRUMENTAL, NATURAL, and PERSONAL errors. First, we will discuss these errors, and then, later, we will explain the common mistakes in transit work.

Identifying Instrumental Errors

A transit will not measure angles accurately unless the instrument is in the following condition:

1. The vertical cross hair must be perpendicular to the horizontal axis. If the vertical cross hair is not perpendicular, the measurement of horizontal angles will be inaccurate.
2. The axis of each of the plate levels must be perpendicular to the vertical axis. If they are not, the instrument cannot be accurately leveled.

If the instrument is not level, the measurement of both horizontal and vertical angles will be inaccurate.

3. The line of sight through the telescope must be perpendicular to the horizontal axis. If it is not, the line of sight through the telescope inverted will not be 1800 opposite the line of sight through the telescope erect.

4. The horizontal axis of the telescope must be perpendicular to the vertical axis. If it is not, the measurement of both horizontal and vertical angles will be inaccurate.

5. The axis of the telescope level must be parallel to the line of sight through the telescope. If it is not, the telescope cannot be accurately leveled. If the telescope cannot be accurately leveled, vertical angles cannot be accurately measured.

6. The point of intersection of the vertical and horizontal cross hairs must coincide with the true optical axis of the telescope. If it doesn't, measurement of both horizontal and vertical angles will be inaccurate.

NOTE: Any or all of the above conditions may be absent in an instrument that is defective or damaged, or one that needs adjustment or calibration.

Identifying Natural Errors

Common causes of natural errors in transit work are as follows:

1. Settlement of the tripod in yielding soil. If the tripod settled evenly—that is, if the tip of each leg settled precisely the same amount—there would be little error in the results of measuring horizontal angles. Settlement is usually uneven, however, which results in the instrument not being level.

2. Refraction—but the effect of this is usually negligible in ordinary precision surveying.

3. Unequal expansion or contraction of instrument parts caused by excessively high or low temperature. For ordinary precision surveying, the effect of this is also usually negligible.

4. High wind may cause plumbing errors when you are plumbing with a plumb bob and cord and may also cause reading errors because of vibration of the instrument.

Identifying Personal Errors

Personal errors are the combined results of carelessness and of the limitations of the human eye in setting up and leveling the instrument and in making observations.

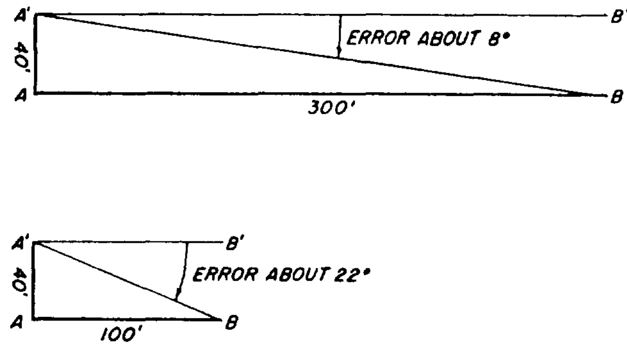


Figure 13-28.-Exaggerated illustration of error caused when the transit is not centered exactly over the occupied station.

Common causes of personal errors in transit work are as follows:

1. Failure to plumb the vertical axis exactly over the station. Figure 13-28 shows how the result of inaccuracy increases drastically as the sight distance decreases. In that figure, an instrument supposed to be set up at A was actually set up at A´, 40 ft away from A. (For demonstration purposes the figure was exaggerated to magnify the error; in actual practice the eccentricity amounts only to a fraction of an inch. Remember that mathematically, 1 in. is the arc of 1 min when the radius is 300 ft.)

In the upper view, you can see that with B located 300 ft from A, the angular error caused by the displacement is about 8°. In the lower view, however, with B located only 100 ft from A, the angular error caused by the displacement is about 22°.

The practical lesson to be learned from this is that you must plumb the instrument much more carefully for a short sight than for a long one.

2. Failure to center plate level bubbles exactly. The result of this is that the instrument is not leveled exactly. The consequent error is at a minimum for a horizontal sight and increases as a sight becomes inclined.

The practical lesson is that you should level the instrument much more carefully for an incline sight than for a horizontal one.

3. Inexact setting or reading of a vernier. The use of a small, powerful pocket magnifying glass is helpful here. Also, when you have determined the vernier graduation that most nearly coincides with a limb graduation, it is a good idea to check your selection by examining the graduations on either side of the one selected. These should fall in coincidence with the limb counterparts by about the same amount.

4. Failure to line up the vertical cross hair with the true vertical axis of the object sighted. The effect is similar to that of not plumbing exactly over the station, which means that the error increases drastically as the length of the sight decreases.

5. Failure to bring the image of the cross hair or that of the object sighted into clear focus (parallax). A fuzzy outline makes exact alignment difficult.

Common mistakes in transit work are the following:

1. Turning the wrong tangent screw. For example, by turning the lower tangent screw AFTER taking a backsight, you will introduce an error in the backsight reading.

2. Forgetting to tighten the clamp(s), or a clamp slipping when it is supposed to be tight.

3. Reading in the wrong direction from the index (zero mark) on a double vernier.

4. Reading the wrong vernier; for example, reading the vernier opposite the one that was set.

5. Reading angles in the wrong direction; that is, reading from the outer row rather than the inner row, or vice versa, on the horizontal scale.

6. Failure to take a full-scale reading before reading the vernier. For example, you may drop 20 to 30 min from the reading, erroneously recording only the number of minutes indicated on the vernier, such as 15°18´ instead of 15°48´. Do not be so intent on reading the vernier that you lose track of the full-scale reading of the circle.

CARING FOR AND MAINTAINING SURVEYING INSTRUMENTS

The accuracy and quality of a survey depend upon the condition of the surveying instrument and the experience of the surveyor. The life expectancy and usefulness of an instrument can be extended considerably by proper and careful handling, stowing, and maintenance. Undoubtedly, by simply working in your rating conscientiously, you will become experienced in the proper use of the instrument.

As stated earlier, every instrument is accompanied by an instruction manual that tells you not only the proper operation and components of the instrument but also procedures for its proper care and maintenance. Study this instruction manual thoroughly before you even attempt to use the instrument.

Carrying and Stowing

Every transit, theodolite, or level comes equipped with a carrying box or case. The instrument and its accessories can be stowed in the case in a manner that ensures a minimum of motion during transportation. The instrument should ALWAYS be stowed in the carrying case when it is not in use.

Cleaning and Lubricating

In general, all surveying instruments, equipment, or tools must be cleaned thoroughly immediately after you have used them. For example, you dust off the transit or theodolite and wipe it dry before placing it back in its case after each use. Remove all dust with a clean cloth. This applies particularly to the optical parts. Chamois leather is suitable for this purpose, but it is better to use a clean handkerchief than a soiled chamois leather. Use no liquid for cleaning — neither water, petrol, nor oil. If necessary, you can breathe on the lenses before polishing them. When the instrument becomes wet, you should remove its case and dry it thoroughly at room temperature as soon as you get home. If you leave the instrument in the closed case, the air inside the hood will take up humidity by increasing temperature and will in time diffuse inside the instrument. While cooling off, the water will condense and form a coating or tarnish that may make any sighting with the telescope and reading of the circles difficult.

Remove any mud or dirt that may adhere to the tripod, range pole, level rod, and so forth, after each use. Clean each instrument, equipment, or tool after each use to eliminate the chance of forgetting it. This is important, especially when the surveying gear is made of a material that is susceptible to rust action or decay.

When lubricating the instruments, you must use the recommended lubricant for each part in conjunction with the climatic condition in your area. For instance, it is recommended that graphite be used to lubricate transit moving parts when the transit is to be used in sub-zero temperatures instead of the light film of oil (preferably watch oil) when its use is confined to an area with normal weather conditions. The lubricant should be applied thinly to avoid making the lubricated parts an easy repository for dust or catcher of dust.

Consult the manufacturer's manual or your senior EA whenever you are in doubt before doing anything to an instrument.

NOTE: Information on tests, adjustments, and minor repairs of surveying instruments will be presented at the EA2 level.

TRAVERSE OPERATIONS (FIELD PROCEDURES)

A survey traverse is a sequence of lengths and directions of lines between points on the earth, obtained by or from field measurements and used in determining positions of the points. A survey traverse may determine the relative positions of the points that it connects in series; and, if tied to control stations based on some coordinate system, the positions may be referred to that system. From these computed relative positions, additional data can be measured for layout of new features, such as buildings and roads.

Traverse operations (actions commonly called TRAVERSING) are conducted for basic area control; mapping; large construction projects, such as military installation or air bases; road, railroad, and pipeline alignment; control of hydrographic surveys; and for many other projects. In general, a traverse is always classified as either a CLOSED TRAVERSE or an OPEN TRAVERSE.

A closed loop traverse (fig. 13-29, view A), as the name implies, forms a continuous loop, enclosing an area. This type of closed traverse starts and ends at the same point, whose relative horizontal position is known. A closed connecting traverse (fig. 13-29, view B) starts and ends at separate points, whose relative positions have been determined by a survey of an equal or higher order accuracy. An open traverse (fig. 13-29, view C) ends at a station whose relative position is not previously known, and unlike a closed traverse, provides no check against mistakes and large errors. Open traverses are often used for preliminary survey for a road or railroad.

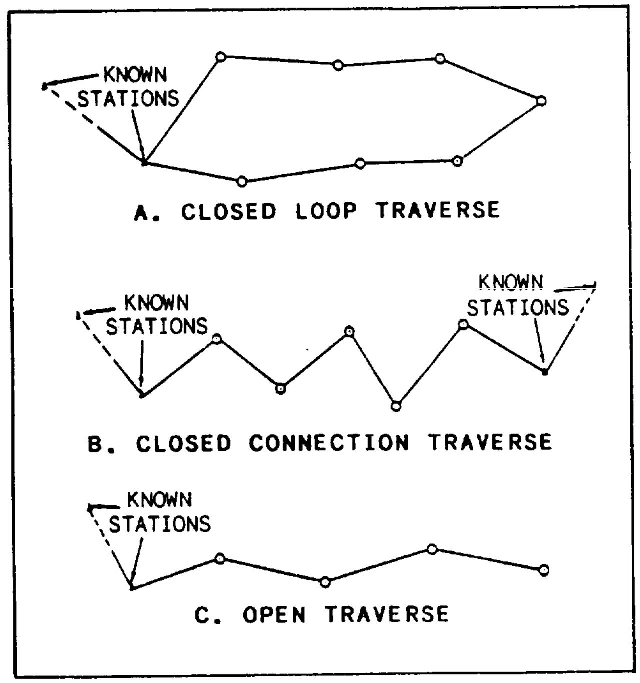


Figure 13-29.-Types of traverses.

The order of ACCURACY for any traverse is determined by the equipment and methods used in the traverse measurements, by the accuracy attained, and by the accuracy of the starting and terminating stations. Hence, the order of accuracy must be specified before the measurements are started. For engineering and mapping projects, the distance measurement accuracy for both electronic and taped traverses for first, second, and third order are 1/35,000, 1/15,000, and 1/7,500, respectively.

For military use such as field artillery, lower order accuracies of fourth, fifth, and sixth are 1/3,000, 1/1,000, and 1/500, respectively. The order referred to as lower order is applied to all traverses of less than third order.

To accomplish a successful operation, the traverse party chief must ensure that initial preparations and careful planning are done before the actual traversing begins. In the remainder of this chapter, we will discuss some of the basic procedures normally undertaken by a transit-tape traverse party.

ORGANIZING THE PARTY

A traverse party may vary from 2 to about 12 personnel, all under the supervision of a traverse party chief. It usually consists of a distance-measuring crew, an angle crew, sometimes a level crew, and other support personnel. This breakdown of personnel is ideal; but, on many occasions, the same personnel will have to perform a variety of tasks or functions. Therefore, each party member is trained to assume various duties and functions in several phases of the work survey.

CONDUCTING A RECONNAISSANCE

Whenever possible, a reconnaissance must be made to determine the starting point, the route to be followed, the points to be controlled, and the closing station. When selecting the starting and closing points, you must select an existing control station that was determined by a survey whose order of accuracy was equal to or greater than the traverse to be run. When running a traverse in which the direction of the traverse lines are not fixed before the start, select a route that offers minimum clearing of traverse lines. The best available maps and aerial photographs should be used during the office and field reconnaissance. By selecting a route properly, you can lay out the traverse to pass relatively close to points that have to be located or staked out.

On other surveys, such as road center line layout, the directions of the traverse lines are predetermined, and all obstructions, including large trees, have to be cleared from the line. Often the assistance of the equipment and construction crews is needed at this point. For the lower order surveys and where taping is used, the exact route and station locations normally are selected as the traverse progresses. These stations have to be selected so that at any one station, both the rear and forward stations are visible, and only a minimum number of instrument setups is kept, reducing the possibility of instrument error and the amount of computing required.

Furthermore, the electronic distance-measuring devices (EDMs) have made traverse reconnaissance even more important. The possibility y of using an EDM should be considered after the general alignment in direction and the planned positioning of stations. A tower or platform installed to clear surface obstruction will permit comparatively long optical sights and distance measurements, hence avoiding the necessity of taping it in short increments.

PLACING STATION MARKS

Some station marks are permanent markers, and some are temporary markers, depending upon the purpose of the traverse. A traverse station that will be reused over a period of several years is usually marked in a permanent manner. Permanent traverse station markers are of various forms, including such forms as an iron pipe filled with concrete; a crosscut in concrete or rock; or a hole drilled in concrete or rock and filled with lead, with a tack to mark the exact reference point. Temporary markers, on the other hand, are used on traverse stations that may never be reused, or perhaps will be reused only a few times within a period of 1 or 2 mo. Temporary traverse station markers are usually 2-in. by 2-in. wooden hubs, 12 in. or more in length. They are driven flush with the ground and have a tack or small nail on top to mark the exact point of reference for angular and linear measurements. To assist in recovering the hub, a 1-in. by 2-in. wooden guard stake, 16 in. or more in length is driven at an angle so that its top is about 1 ft over the hub. Keel (lumber crayon) or a large marking pen is used to mark letters and/or numbers on the guard stake to identify the hub. The marked face of the guard stake is toward the hub. Since many of the hubs marking the location of road center lines, landing strips, and other projects will require replacement during construction, reference marks are placed several hundred feet or meters away from the station they reference. Reference marks, usually similar in construction to that of the station hub, are used to reestablish a station if its marker has been disturbed or destroyed.

NOTE: Procedures for marking hub and guard stakes for traverse stations, road center line layout, and other surveys are presented in the next chapter.

TYING IN TO EXISTING CONTROL

As we discussed earlier in this chapter, the starting point of a closed traverse must be a known position or control point; and, for a closed loop traverse, this point is both the starting and closing point. Closed connecting traverses start at one control point and tie into another control point.

A traverse starting point should be an existing station with another station visible for orienting the new traverse. The adjacent station must be intervisible with the starting point to make the tie easy. If you do not find the adjacent station easily, you should observe an astronomic azimuth to orient the starting line, and then continue the traverse. Any existing control near the traverse line should be tied in to the new work.

PERFORMING LINEAR MEASUREMENTS

As traversing progresses, linear measurements are conducted to determine the distance between stations or points. Generally, the required traverse accuracy will determine the type of equipment and the method of measuring the distance. For the lower orders, a single taped distance is sufficient. However, as the order of accuracy gets higher, DOUBLE TAPING (once each way) is required. Ordinary steel tapes must be compared to an Invar or Lovar tape at specified intervals. For the highest accuracy, electronic distance-measuring devices (EDM) are used to measure linear distances. Linear measurements may also be made by indirect methods, using an angle measuring instrument, like the transit or theodolite with stadia. When the distances are determined by stadia readings, the vertical angles are read and used to convert slope distances to horizontal distances.

If double taping or chaining is required, follow these procedures:

1. Follow a direct line between stations, using a guide, such as a transit and a range pole, for alignment. Start measuring from the occupied station, keeping the front end of the tape aligned with the forward station.

2. Start back from the forward station, using the same alignment but not the same taping points. The second measurement must be independent of the first.

3. Compare the two distances, and if within accuracy requirements, the distance is accepted. If the two measurements disagree by more than the allowable amount, retape the distance.

4. Proceed to the next line measurement, and continue double taping until the tie-in control point is reached.

PERFORMING ANGULAR MEASUREMENT

Horizontal angles formed by the lines of each traverse station determine the relative directions of the traverse lines. These angles are measured using a transit or a theodolite, or determined graphically with a plane table and alidade. In a traverse, three traverse stations are significant: the REAR STATION, the OCCUPIED STATION, and the FORWARD STATION (fig. 13-30). The rear station is that station from which the crew performing the traverse has just moved, or it is a point, the azimuth to which is known. The occupied station is the station at which the crew is now located and over which the surveying instrument is set. The forward station is the next station in succession and constitutes the immediate destination of the crew. The stations are numbered consecutively starting at Number 1 and continuing throughout the traverse. In addition to the number of station, an abbreviation indicating the type of traverse is oftentimes

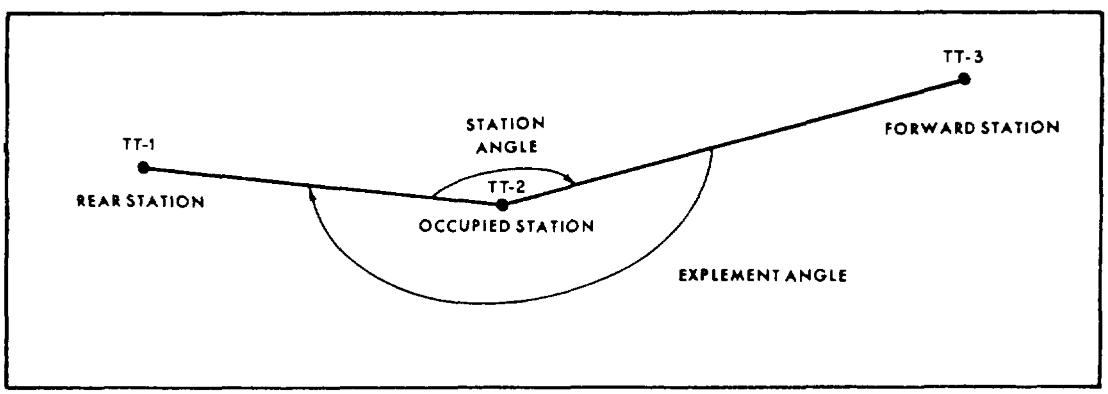


Figure 13-30.-Traverse stations and angles.

included; for example, ET for electronic traverse or TT for theodolite- or transit-tape traverse.

Horizontal angles are always measured at the occupied station by pointing the instrument toward the rear station and turning the angle clockwise to the forward station for the direct angle, and clockwise from the forward to the rear station for the explement (fig. 13-31). If a deflection angle is to be used, plunge the instrument telescope, after sighting the rear station, and read the angle left or right of the forward station.

NOTE: Office procedures for traverse computations and adjustments, methods of computing traverse area, and plotting horizontal control are discussed at the EA2 level.

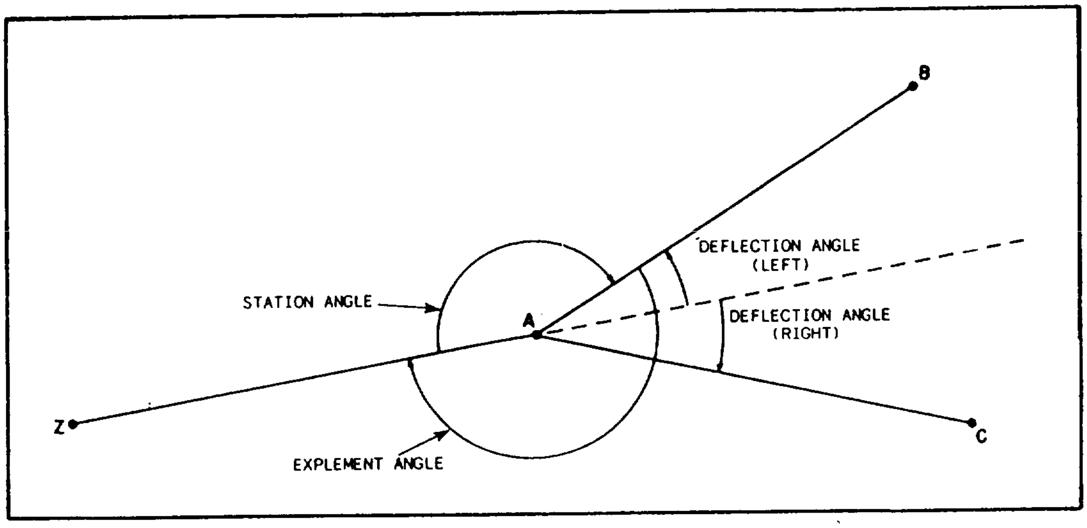


Figure 13-31.-Kinds of angles measured at the occupied station.